eGN Online www.geotechnicalnews.com

Volume 34 • Number 2 • June 2016

GEOTECHNICAL news

Wimbleball dam, England

RST Instruments Ltd. offers 2 Wireless Data Collection Systems to quickly get you connected to your data: **RSTAR** and **DT LINK**. Both systems offer **minimum per channel cost, extra long battery life** and **long distance data transmission.**

WIRELESS DATA COLLECTION

for Geotechnical Monitoring Instrumentation

FULLY AUTOMATED COLLECTION (REMOTELY)

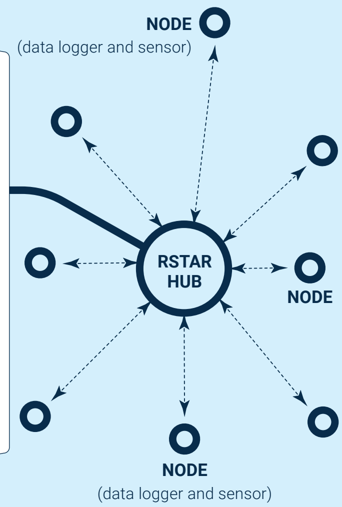
An RSTAR System uses data loggers (nodes) at the sensor level, deployed in a star topology from an active RSTAR Hub containing an RST flexDAQ Data Logger.

rstar
ARRAY RADIO SERIES

FEATURES

Up to 7 years of battery life from 1 lithium 'C' or 'D' cell.
Up to 14 km range from Hub to Node in open country. (depending on antenna type)
Up to 255 nodes per RSTAR Hub.
Based on 900 MHz and 2.4 GHz spread spectrum band. (country dependent)

The RSTAR Hub shown left contains a flexDAQ Data Logger System with an antenna and battery. Collected data is saved to the flexDAQ memory where users can access it remotely, either on-site or off-site.



Watch the video for both systems at: www.rstinstruments.com/Wireless-Data-Collection.html

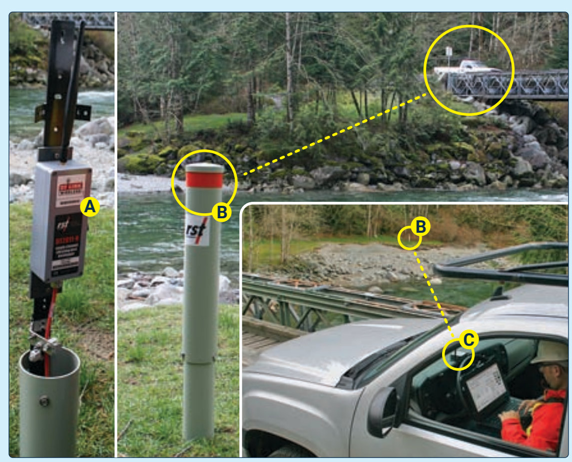
SEMI-AUTOMATED COLLECTION (ON-SITE)

DT LINK is an on-site wireless connection to RST data loggers for quick data collection. Ideal for hard to access areas where the data logger is within line of sight.

DT LINK WIRELESS

FEATURES

Safely & easily collect data from data loggers that are in areas with poor access, trespass issues and hazardous obstacles.
Years of battery life from 1 lithium 'C' or 'D' cell.
Range up to 800 m (900 MHz) and up to 500 m (2.4 GHz).
Collect data in seconds with a laptop connected to DT LINK HUB.



Pictured: (A) DT LINK WIRELESS data logger, connected to a vibrating wire piezometer and housed in a (B) protective enclosure, has its data collected from a laptop connected to the (C) DT LINK HUB - all within seconds from the convenience of your vehicle.

RST's "DT Series" Data Loggers accommodate the RSTAR and DT LINK WIRELESS Systems. Compatible sensor types include:

Vibrating Wire, MEMS, 4-20mA Transmitters and Thermistors.

rst INSTRUMENTS
innovation in geotechnical instrumentation

info@rstinstruments.com
www.rstinstruments.com

CANADA: 604 540 1100
USA: 831 899 1632 or 303 993 9230
EUROPE / MIDDLE EAST / AFRICA: +44 7469 256642
RST Instruments Ltd., 11545 Kingston St., Maple Ridge, BC Canada V2X 0Z5

Certified System ISO 9001

YouTube www.youtube.com/user/RSTgeotechnical
LinkedIn www.linkedin.com/company/rst-instruments-ltd-

GEOTECHNICAL news

PUBLISHER John W. Gadsby

MANAGING EDITOR Lynn Pugh

Editors

Robert Chapuis
John Dunnicliff
Jonathan Fannin
Paolo Gazzarrini
Don Lewycky
Ward Wilson

Managing Editors and Advertising

BiTech Publishers Ltd.
103 - 11951 Hammersmith Way
Richmond, British Columbia
Canada V7A 5H9
tel 604-277-4250 • fax 604-277-8125
email *gn@geotechnicalnews.com*
web *www.geotechnicalnews.com*

GEOTECHNICAL NEWS is published quarterly.

Paper subscription rates:

- within North America: $58.00 CDN per year
- overseas: $85.00 US per year through BiTech Publishers Ltd.

Electronic version:
GEOTECHNICAL NEWS is also available in electronic version. For details,visit *www.geotechnicalnews.com*

Canadian Editorial Office

Canadian Geotechnical Society

Don Lewycky, Editor, CGS News • email: *don.lewycky@edmonton.ca*

Membership Information : Canadian Geotechnical Society

Gibson Group Association Management

Wayne Gibson, 8828 Pigott Road • Richmond, BC V7A 2C4 • tel: 604-277-7527 • email: *cgs@cgs.ca*

United States Editorial Office

Geo-Institute of the American Society of Civil Engineers

Linda R. Bayer, 1801 Alexander Bell Drive, • Reston, VA 20191-4400 • tel: 703-295-6352
fax: 703-295-6351 • email: *lbayer@asce.org*

Editors

Groundwater

Robert P. Chapuis, Dept. CGM, Ecole Polytechnique, PO Box 6079, Sta. CV Montréal, QC, H3T 1J4
tel: 514-340-4711 • fax: 514-340-4477, • email : *robert.chapuis@polymtl.ca*

Instrumentation

John Dunnicliff, Little Leat, Whisselwell, Bovey Tracey, Devon TQ13 9LA, England
tel: +44 1626-832919 • email: *john@dunnicliff.eclipse.co.uk*

Geosynthetics

Jonathan Fannin, Professor of Civil Engineering, University of British Columbia, 6250 Applied Science Lane, Vancouver, BC, V6T 1Z4 • tel: 604-822-3557 • email: jonathan.fannin@ubc.ca

The Grout Line

Paolo Gazzarrini, 11-2246 Folkestone Way, West Vancouver, BC, V7S 2X7 • tel: 604-913-1022
fax: 604-913-0106 • email: *paolo@paologaz.com*

Waste Geotechnics

G. Ward Wilson, Professor, Geotechnical and Geoenvironmental • University of Alberta, Dept. of Civil & Environmental Engineering, 3-069 NREF, Edmonton, AB T6G 2W2 • tel: 780-492-2534
fax: 780-492-8198 • email: *wwilson2@ualberta.ca*

Printed in Canada

Volume 34 • Number 2 • June 2016

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Message from the President

Doug VanDine, President of Canadian Geotechnical Society

Before I tell you what's coming up and what's been going on in the CGS, I would like to talk about "history"; specifically, Canadian geotechnical history and the history of the Canadian Geotechnical Society. My bachelor's degree was in Geological Engineering, where I learned that the first principle of geology is "the processes of the past are the key to the future". Therefore I believe it's important for all of us to know something about our geotechnical history.

In the early 1980s, I served on a forerunner of the CGS Heritage Committee with **Jack Clark** (CGS President 1979-1980), **Dave Devenny** (CGS President 1985-1986) and **Dave Townsend**. At that time I was the "young guy" on the committee. We started to document, both in written form and by recorded interviews, the early geotechnical history of this country and the contributions of the early Canadian geotechnical pioneers. The work of the Heritage Committee has continued and the scope has expanded under the Chairs **Jim Graham** (CGS President 1997-1998 and CGS Secretary General 1999-2007), **Mustapha Zergoun**, **Suzanne Powell** and currently **Dave Cruden**.

One of the more active CGS committees, the Heritage Committee has collected a great deal of Canadian geotechnical history that is housed "virtually" on the CGS web page at ***http://www.cgs.ca/heritage-archive.php?lang=en***. Currently the sub-pages include "Heritage Lectures", "Photographic Collections", "Information and Location for Archived Records", "Lives Lived" and "Recommended Reading". These headings certainly

don't do full justice to what can be found within the sub-pages. There is far too much good information to list here, so I recommend you go online and poke around a little…or a lot. I guarantee you won't be disappointed. Learning at least a little about our past will help you be a better geotechnical professional in the future.

Next year, the CGS will celebrate its 70th anniversary and will hold its 70th Annual Conference in Ottawa in the early fall (**GeoOttawa 2017**). Ottawa is where the CGS began in 1947 as the National Research Council's "Associate Committee on Soil and Snow Mechanics", later called the "Associate Committee on Geotechnical Research". To mark this 70th anniversary, the CGS Heritage Committee is updating the CGS history that was first written by **Bill Eden** in 1985. I'm looking forward to both the conference and the updated history.

Before we get to 2017, this fall the CGS will be hosting the 69th CGS Annual Conference in Vancouver (**GeoVancouver 2016**), immediately preceded by the 5th Canadian Young Geotechnical Engineers and Geoscientists Conference (**cYGEGC 2016**) in Whistler, B.C. (registration is now open at ***www.geovancouver2016.com/*** and ***cygegc2016.com/index.php/en/welcome/***, respectively). I know the organizing committees of both events have been working extremely hard to make these two conferences something for you to remember, both technically and socially.

This spring, **Dr. Antonio Gens** of the University of Barcelona, Spain presented the **97th CGS Cross-Canada Lecture Tour (CCLT)** to twelve well-attended CGS Sections across Canada. This fall, **Dr. Ross Boulanger** of the University of California, Davis Campus will present the **98th CGS CCLT**. I would like to thank the **Canadian Foundation for Geotechnique** for funding the travel costs for these lectures and all the local Sections that have hosted or will host the lectures and the lecturers, along with CGS VP Technical **Angela Küpper** and CGS Director of Finance and Administration **Wayne Gibson,** who both worked so hard behind the scenes to make these lectures happen and run so smoothly.

Also this spring, I had the opportunity to attend the **Engineering Institute of Canada's Award Banquet**, where CGS members **Jean-Pierre Tournier** (CP Rail Engineering Medal), **Gordon Fenton** (FEIC) and **David Woeller** (FEIC) were awarded their honours. These very well-deserving CGS members only received these honours because someone in the CGS nominated them. I ask you to look elsewhere in this issue of *CGS News* for details on to how you can nominate one of your CGS colleagues for the 2017 EIC Awards and Fellowships. Nominations are due at CGS Headquarters by **July 15, 2016**.

As usual, over the past few months, I've been helped so much by CGS Executive Director **Michel Aubertin**, as well as **Wayne Gibson** and **Lisa McJunkin** at CGS Headquarters.

Until next time,

Provided by Doug VanDine – President - 2015/2016

Message du président

Avant que je vous dise ce qui ce qui se passe et ce qui s'en vient à la SCG, j'aimerais vous parler d'« histoire », particulièrement de celle de la géotechnique canadienne et de la Société canadienne de géotechnique. Je détiens un baccalauréat en génie géologique, dans le cadre duquel j'ai appris que le premier principe de la géologie est « les processus du passé sont la clé de l'avenir ». Je crois donc qu'il est important pour nous tous de connaître notre histoire géotechnique.

Au début des années 1980, j'ai été membre de l'ancêtre du Comité sur le patrimoine de la SCG avec **Jack Clark** (président de la SCG 1979-1980), **Dave Devenny** (président de la SCG 1985-1986) et **Dave Townsend**. À cette époque, j'étais le « jeune homme » sur le comité. Nous avons commencé à documenter, par écrit et par entrevues enregistrées, les débuts de l'histoire de la géotechnique de ce pays et les contributions des premiers pionniers canadiens de la géotechnique. Le travail du Comité sur le patrimoine s'est poursuivi, et sa portée s'est élargie sous la direction de **Jim Graham** (président de la SCG 1997-1998 et secrétaire général de la SCG 1999-2007), de **Mustapha Zergoun**, de **Suzanne Powell** et, actuellement, de **Dave Cruden**.

L'un des comités les plus actifs de la SCG, le Comité sur le patrimoine, a recueilli plusieurs éléments sur l'histoire de la géotechnique canadienne qui sont conservés « virtuellement » sur le site Web de la SCG, à http://www.cgs.ca/heritage-archive.php?lang=fr. Actuellement, les sous-pages comprennent « Conférences patrimoniales », « Collections de photos », « Information sur les archives et leur emplacement », « Notices nécrologiques » et « Lectures recommandées ». Ces titres ne rendent certainement pas complètement justice à ce qui peut se trouver sur ces sous-pages. Il y a beaucoup trop d'éléments d'information intéressants pour les mentionner tous ici. Je vous recommande donc d'aller en ligne et de fureter un peu... ou beaucoup. Je vous garantis que vous ne serez pas déçu. En apprendre au moins un peu sur notre passé vous aidera à être un meilleur professionnel de la géotechnique à l'avenir.

L'année prochaine, la SCG célébrera son 70e anniversaire et tiendra sa 70e conférence annuelle à Ottawa, au début de l'automne (**GéoOttawa 2017**). La SCG a vu le jour à Ottawa en 1947, comme l'« Associate Committee on Soil and Snow Mechanics » du Conseil national de recherche, plus tard appelé le « Comité associé de recherches géotechniques ». Pour marquer ce 70e anniversaire, le Comité sur le patrimoine de la SCG actualise l'histoire de la SCG qui a été initiale-

ment rédigée par **Bill Eden** en 1985. J'ai hâte d'assister à la conférence et de consulter cette histoire actualisée.

Avant d'arriver en 2017, la SCG organisera cet automne sa 69e conférence annuelle à Vancouver (**GéoVancouver** 2016), précédée immédiatement par la 5e Conférence canadienne des jeunes géotechniciens et géoscientifiques (**cYGEGC** 2016), à Whistler, en C.-B. (L'inscription à ces conférences est maintenant commencée, respectivement sur ***http://fr.geovancouver2016.com/*** et sur ***http://cygegc2016.com/index.php/fr/bienvenue/***.) Je sais que les comités organisateurs de ces deux événements travaillent très fort pour que ces deux conférences soient mémorables, tant sur le plan technique que social.

Ce printemps, le **Dr Antonio Gens** de l'Université de Barcelone, en Espagne, a présenté la **97e Tournée de conférences transcanadiennes de la SCG (TCT)** à 12 sections de la SCG; de nombreuses personnes ont assisté à ces présentations. Plus tard cet automne, le **Dr Ross Boulanger** de l'Université de la Californie à Davis présentera la **98e** TCT de la SCG. J'aimerais remercier la **Fondation canadienne de géotechnique** qui finance les frais de déplacement pour ces conférences et toutes les sections locales qui ont accueilli ou qui accueilleront les conférences et les conférenciers, ainsi que la v.-p. technique de la SCG, **Angela Küpper** et le directeur des finances et de l'administration de la SCG, **Wayne Gibson**, qui ont travaillé très fort en arrière scène pour que ces conférences aient lieu et se déroulent aussi bien.

Également ce printemps, j'ai eu l'occasion d'assister au **banquet de remise des prix de l'Institut canadien des ingénieurs**, où les membres de la SCG, **Jean-Pierre Tournier** (Médaille CP Rail Engineering), **Gordon Fenton** (FICI) et **David Woeller** (FICI) ont reçu leurs distinctions. Ces membres de la SCG qui le méritaient vraiment bien n'ont reçu ces distinctions que parce que quelqu'un de la SCG, a présenté leur candidature. Je vous demande de consulter le présent numéro de *CGS News* pour de plus amples détails sur la façon dont vous pouvez soumettre la candidature d'un de vos collègues de la SCG pour les prix et les titres de Fellow de l'ICI pour 2017. Les candidatures doivent être envoyées au siège social de la SCG d'ici le **15 juillet** 2016.

Comme d'habitude, au cours des derniers mois, j'ai été beaucoup aidé par le directeur général de la SCG, **Michel Aubertin**, ainsi que par **Wayne Gibson** et **Lisa McJunkin**, au siège social de la SCG.

À la prochaine!

Fourni par Doug VanDine – Président 2015/2016

From the Society

Call for Nominations for 2017 Awards and Fellowships

Engineering Institute of Canada (EIC)

Award of Honour	Brief Description/Comments
Sir John Kennedy Medal	For outstanding service to the profession or for noteworthy contributions to the science of engineering, or to the benefit of the EIC. EIC's most distinguished award (given every two years)
Julian Smith Medal	For achievement in the development of Canada
John B. Stirling Medal	For leadership and distinguished service at the national level within the EIC and/or its member societies
CP Rail Engineering Medal	For leadership and service at the regional, branch and section levels by members of EIC member societies
K.Y. Lo Medal	For significant engineering contributions at the international level, such as promotion of Canadian expertise overseas; training of foreign engineers; significant service to international engineering organizations; and advancement of engineering technology recognized internationally
Fellowship of the EIC	For excellence in engineering and services to the profession and to society
Honorary Member	For non-members of the EIC and its member societies, and on occasion non-engineers, who have achieved outstanding distinction through service to engineering and the profession of engineering in Canada

Prix ou distinction	Courte description/Commentaires
Médaille Sir John Kennedy	En reconnaissance de services exceptionnels rendus à la profession d'ingénieur, ou des contributions remarquables à la science de l'ingénierie, ou au bénéfice de l'Institut. Plus prestigieux prix de l'ICI; décerné tous les deux ans.
Médaille Julian Smith	En reconnaissance des réalisations dans le développement du Canada; jusqu'à deux médailles remises chaque année.
Médaille John B. Stirling	En reconnaissance du leadership et des services rendus à l'échelle nationale à l'Institut ou à ses Sociétés Membres.
Médaille CP Rail Engineering	En reconnaissance de nombreuses années de leadership et de service par les membres des sociétés au sein de l'Institut aux niveaux régional (direction ou section); jusqu'à deux médailles remises chaque année.
Médaille K.Y. Lo	Pour des contributions remarquables au domaine de l'ingénierie au niveau international, comme la promotion de l'expertise canadienne à l'étranger, la formation d'ingénieurs étrangers, un service exceptionnel rendu à des organisations d'ingénierie internationales et l'‘avancement d'une technologie d'ingénierie reconnu sur la scène internationale.
Titre de Fellow	Pour l'excellence en ingénierie et des services rendus à la profession et à la société.
Membre honoraire	Pour les non-membres de l'ICI et de ses sociétés membres, et occasionnellement pour des personnes qui ne sont pas des ingénieurs, qui se méritent cette remarquable distinction en raison de services rendus au domaine de l'ingénierie et à la profession de l'ingénierie au Canada.

As a constituent Society of the **Engineering Institute of Canada** (EIC), CGS members are eligible for awards and fellowships of the EIC which are summarized below. CGS members are encouraged to submit EIC nominations of fellow members to CGS Headquarters by **July 15, 2016.**

Nominations must include:

1. a completed EIC Nomination Form which is available from ***http://eic-ici.ca/honours_awards/***)
2. a nomination letter
3. supporting letters from colleagues, preferably Fellows of the EIC (FEIC).

Past CGS member recipients of EIC Awards and Fellowships can be found on the CGS website ***www.cgs.ca/awards.php?lang=en.*** It is recommended that nominators review the awards details and criteria prior to preparing nominations. For more information contact CGS Headquarters at:

The Canadian Geotechnical Society
8828 Pigott Road
Richmond, BC
V7A 2C4, Canada,
Fax: (604) 277-7529
E-mail: ***admin@cgs.ca***

Appel de candidatures pour les prix et médailles 2017 de l'Institut canadien des ingénieurs (ICI)

À titre de société membre de l'**Institut canadien des ingénieurs** (ICI), les membres de la SCG sont admissibles aux prix et médailles de l'ICI décrits ci-dessous. Les membres de la SCG sont encouragés à soumettre des candidatures de collègues membres pour l'ICI au siège social de la SCG d'ici le **15 juillet 2016**.

Les candidatures doivent inclure :

1. un formulaire de candidature de l'ICI dûment rempli qui est disponible sur le site ***http://eic-ici.ca/honours_awards/***);
2. une lettre de mise en candidature;
3. des lettres de recommandation de collègues, préférablement des fellows de l'ICI.

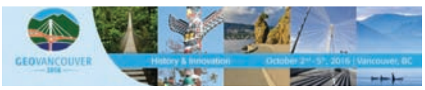
Il est recommandé que les personnes qui soumettent des candidatures examinent les détails et les critères des prix (Fellow et Médailles) avant de les préparer. Pour obtenir de plus amples renseignements, communiquez avec le siège social de la SCG à :

La Société canadienne de géotechnique
8828 Pigott Road
Richmond, C.-B.
V7A 2C4, Canada
Télécopieur : 604-277-7529
Courriel : ***admin@cgs.ca***

Les noms des membres de la SCG qui ont déjà reçu des prix et des bourses de recherche de l'ICI sont affichés sur le site Web de la SCG à ***www.cgs.ca/awards.php?lang=fr***.

Upcoming Conferences and Seminars

69th Canadian Geotechnical Conference
October 2 to 5, 2016
Vancouver, British Columbia, Canada



The **Vancouver Geotechnical Society** and the Canadian Geotechnical Society invite you to the **69th Canadian Geotechnical Conference**. The conference will be held from October 2nd to 5th, 2016 in Vancouver, British Columbia, Canada. It will cover a wide range of topics, including specialty sessions that are of local and national relevance to the disciplines of geotechnical and geo-environmental engineering. In addition to the technical program and plenary sessions, the conference will include a complement of short courses, technical tours, local excursions and entertaining social activities.

The official languages for the conference will be English and French. Vancouver is well known for its beautiful scenery, which encompasses the Coast Mountains, the Fraser River Delta and the Strait of Georgia. The city has been host to many national and international events, including the 2010 Winter Olympics. This breathtaking surrounding lends itself to a wide variety of geological conditions and geotechnical challenges, including high seismicity, steep terrain and soft soils.

The conference will be held at the picturesque Westin Bayshore Hotel which is well situated between the downtown business district and Stanley Park.

The theme of the conference is **History and Innovation**, which will recognize the historical achievements and lessons learned over time while highlighting innovation in geotechnical engineering research and practice.

Please address any questions to the conference co-chairs: **Mustapha Zergoun** at ***mzergoun@thurber.ca*** **Andrea Lougheed** at ***alougheed@thurber.ca***, or the Conference Secretariat at ***secretariat@geovancouver2016.com*** The conference website is ***www.geovancouver2016.com***.

Quesnel Bridge

69e conférence canadienne de géotechnique
2 - 5 octobre 2016,
Vancouver, Colombie Britannique, Canada

La Société géotechnique de Vancouver et la Société canadienne de géotechnique vous invitent à participer à **GéoVancouver 2016**; il s'agit de la 69e conférence canadienne de géotechnique. La conférence se déroulera du 2 au 5 octobre 2016 à Vancouver, Colombie Britannique, Canada. Elle couvrira un large spectre de thèmes incluant des séances spéciales d'intérêt local et national dans les domaines de la géotechnique et géoenvironmental. En plus du programme technique et des séances plénières, la conférence inclura des cours intensifs, des visites techniques, des excursions guidées et des activités sociales amusantes.

Les langues officielles de la conférence seront le français et l'anglais. Vancouver est bien connue pour sa beauté spectaculaire avec les montagnes côtières, le fleuve Fraser et le détroit de Georgia. La ville a été l'hôtesse de nombreux évènements nationaux et internationaux, incluant les Jeux Olympiques d'hiver en 2010. Cette région surprenante comprend une grande variété de conditions géologiques et de défis géotechniques tels qu'une sismicité élevée, des terrains accidentés et des sols mous. La Conférence se tiendra à l'Hôtel Westin Bayshore qui est bien situé, entre le centre-ville d'affaires et le parc Stanley.

Le thème de GéoVancouver 2016 est **Histoire et Innovation** et il vise à reconnaitre les accomplissements historiques et les leçons apprises au fil du temps, tout en mettant en valeur l'innovation dans la recherche et la pratique de la géotechnique.

Vous pouvez acheminer toutes questions aux coprésidents de la conférence: **Mustapha Zergoun** à ***mzergoun@thurber.ca*** ou **Andrea Lougheed** à ***alougheed@thurber.ca*** ou Conférence Secrétariat à ***secretariat@geovancouver2016.com*** ou ***www.geovancouver2016.com***

5th Canadian Young Geotechnical Engineers & Geoscientists Conference
September 29 to October 1, 2016
Whistler, British Columbia

The 5th Canadian Young Geotechnical Engineers & Geoscientists Conference is a triennial Canadian Geotechnical Society event. The Conference is targeted towards young engineers and geoscientists who are looking to exchange technical information with their peers and build meaningful networks in a relaxed, supportive, and motivational environment. The conference will be hosted in Whistler, B.C. from September 29th to October 1st, 2016, prior to GeoVancouver 2016. Participants are encouraged to submit abstracts and prepare short presentations. The conference registration deadline is **June 24, 2016**. For more information go to ***www.cygegc2016.com*** or contact the conference chairs **Julian McGreevy** and **Maraika De Groot** at ***chair@cygegc2016.com***.

3rd International Conference on Performance-based Design in Earthquake Geotechnical Engineering (PBD-III)

Call For Abstracts

July 16 to 19, 2017
Vancouver, British Columbia

The **3rd International Conference on Performance-based Design in Earthquake Geotechnical Engineering (PBD-III)** will be held in Vancouver, BC from July 16 - 19, 2017. The **PBD-III** Conference is organized under the auspices of the International Society of Soil Mechanics and Geotechnical Engineering - Technical Committee TC203 on Geotechnical Earthquake Engineering and Associated Problems (ISSMGE-TC203).

Join an international community of geo-professionals working to share and advance performance-based design practices for geotechnical earthquake engineering across a broad range of civil infrastructure problems. The coverage will be diverse, including case histories and practice-oriented papers, recent research findings, innovative technologies, and the emerging arts from across the world. An international mix of professional engineers, researchers, specialty contractors, educators, and students will interact across a broad range of keynote and theme lectures, technical sessions, short courses, panel discussions, and field trips.

PBD-III Vancouver seeks oral presentations and posters for the conference. All abstracts must be submitted using the on-line submission page on the conference website, using the template provided and must be received before the close of abstract submissions on **July 31, 2016**. At least one author of an accepted paper must register for the conference by March 15, 2017 in order to be invited to make a presentation (oral or poster) in the technical program. Questions regarding sessions, topics and the technical program should be directed to the PBD-III Vancouver Technical Committee at ***techcommittee@pbdiiivancouver.com***.

Everyone involved with **PBD-III** is excited to be hosting this international event and is looking forward to seeing you in Vancouver. The exciting technical and social programs planned are only possible though the hard work and dedication of many individuals, including all the conference committee members, local organizing committee members, and TC203 members. Together, they look forward to organizing a rewarding experience and interacting with our international friends and fellow geo-professionals. For more information please consult the conference website at ***http://www.pbdiiivancouver.com/index.php?***

Members in the News

Appointment of Dr. Norbert Morgenstern as Honorary Professor at Zhejiang University, PRC

Dr. **Norbert Morgenstern**, distinguished university professor (emeritus) of Civil and Environmental Engineering at the University of Alberta was recently appointed Honorary Professor at Zhejiang University, PRC, "In recognition of his scholarship and outstanding achievements". Following a ceremony on March 16, 2016, Professor Morgenstern delivered a lecture on "*The Stability of Earth Structures: Risk and Reliability*".

Submitted by Vivian Giang, MA
University of Alberta Geotechnical Centre

Division News

CGS Engineering Division

Soliciting Input for an Engineering Geology Monograph

As discussed at the GeoRegina and GeoQuebec CGS Engineering Geology Division Executive meetings, the CGS Engineering Geology Division will be pursuing the publication of an **Engineering Geology Monograph** based on the Canadian experience. We would like to solicit input in terms of the content to include as well as suggestions for chapter topics, etc. It is envisioned that the monograph will capture the history, significant events, innovations and contributions of Canadians to the field of engineering geology. We would not like to leave anyone or any significant topic out of this monograph. As such, we are soliciting the CGS membership (and beyond) for their ideas in terms of topics and people to include. If you would like to contribute to a particular chapter of the monograph, please contact me at ***vlach@rmc.ca***. or at (613) 541-6000 x 6398. We require any and all feedback by **August 31, 2016**.

Thank you for your kind consideration and we look forward to your comments.

Submitted by Nicholas Vlachopoulos
Division Chair – Engineering Geology Division

Heritage Committee

Canadian Geotechnical Society Virtual Archives

There are rich but rarely used resources in Canada that consist of files containing historical information on geotechnical laboratory and field research, geotechnical investigations, work of committees and geotechnical expertise. Ways to identify and use these resources have been developed by the Heritage Committee of the Canadian Geotechnical Society in the form of virtual archives on the CGS web site, where the location and content of accessible historical geotechnical material are given.

CGS members and others are invited to submit candidate material for consideration. The submission should give the location of the material, a description of its nature and content, its historical significance and the conditions under which it can be accessed. Do not submit physical archival material as the Society has no space to store it, however electronic copies of photographs or materials are welcome.

Your contribution to the CGS Virtual Archives web page should be sent to the Chair of the CGS Heritage Committee, Dr. **Dave Cruden**, at ***dcruden@ualberta.ca***

History of the Canadian Foundation for Geotechnique: Part 2

Introduction

In Part 1 of this series, published in the December 2015 issue of Geotechnical News, the focus was on the two predecessor organizations of the Foundation, namely the Canadian Geotechnical Fund (CGF) and Geo-Contributions (GC). This second article focuses on the period spanning from 2000, when the Foundation was established, to the present date.

The **Canadian Foundation for Geotechnique** (CFG), also known as "the Foundation", is a registered charitable organization that funds the awards, prizes and distinguished lectures of the **Canadian Geotechnical Society** (CGS), and supports other activities that recognize geotechnical excellence. Over the past three years, the Foundation has disbursed over $30,000 yearly for various awards, prizes and scholarships, and has sponsored the Cross Canada Lecture Tours (CCLTs) together with industry partners.

The Foundation has fostered and recognized excellence in the Canadian geotechnical community for many years, and one could easily take for granted that it will always do so. No organization, however, stands the test of time without the hard work and dedication of a number of individuals. An attempt to recognize them, as well as to highlight major initiatives over the years, is made herein. We apologize for any errors and omissions.

Canadian Foundation for Geotechnique

The **Canadian Foundation for Geotechnique** was the new name proposed to replace **Geo-Contributions (GC)**. The name was approved by the GC Board of Trustees at the Annual General Meeting in October 2000, and accepted by Industry Canada in December 2000 as a charitable organization.

Similar to its predecessors, the Canadian Geotechnical Fund (CGF) and GC, the Foundation's purpose was "To recognize and foster excellence in the geotechnical field in Canada". When registered in 2000 it had the same Officers and Board of Trustees as GC.

The Foundation, by maintaining an adequate funding base through investments and soliciting donations from individuals, local geotechnical groups and corporations, had the following objectives:

- Fund the awards and prizes recommended by the CGS.
- Support the Geotechnical Research Board (GRB) through an annual award provided to the presenter of the Canadian Geotechnical Colloquium.
- Fund the travel costs of the Cross Canada Lecture Tours.
- Establish funding for geotechnical scholarships.
- Support other activities that recognize geotechnical excellence.

As described in Part 1 of this article, the awards and programs established prior to 2000 and currently funded by the Foundation are:

- **R.F. Legget Award**
- **R.M. Quigley Award**
- **Cross Canada Lecture Tours**
- **Canadian Geotechnical Colloquium**
- **R.M. Hardy Keynote Address**
- **Thomas Roy Award**
- **Roger J.E. Brown Award**
- **G. Geoffrey Meyerhof Award**
- **John A. Franklin Award**
- **A.G. Stermac Award**
- **Undergraduate Student Report Awards**
- **Graduate Student Presentation Award**

The CGS subsequently established the following additional awards that the Foundation also now supports:

- **Geosynthetics Award**; established by the CGS in 2000 to recognize an individual or individuals in the application of geosynthetics in civil, geotechnical or geoenvironmental engineering.
- **Geoenvironmental Award**; established by the CGS in 2000 to recognize outstanding geoenvironmental engineering.

As a millennium project in 2000, Dr. **M. Bozozuk**, President of the Foundation designed the **Legget Medal** to replace the R.F. Legget Award. It is made of sterling silver by the Royal Canadian Mint and was presented for the first time to Dr. **D H. Shields** at the CGS Annual Conference in Montreal in October 2000. Past winners of the R.F. Legget Award were also given the opportunity to receive the Medal.

In 2001, the Foundation agreed to contribute $1,500 annually towards the cost of preparing the certificates and plaques awarded by the CGS. In 2003, the CGS and the Foundation increased the honoraria for two Graduate Student Presentation Awards and four Undergraduate Student Report Awards (for individual and group submissions) to a total of $3,750. In 2005 the Foundation contributed, for the first time, $1,000 to support two CGS members to participate in **Young Geotechnical Engineers Conferences**. This amount currently stands at $4,000 and the Foundation is preparing to support two young members to attend the 2017 Conference in Seoul, Korea.

Over the past several years, the Foundation has received generous and excellent support from corporate sponsors to provide travel costs for the Cross Canada Lecture Tours. In 2005, the Foundation established a Cross Canada Lecture Tour Reserve Fund to manage and report on activities related to fund raising revenues and expenditures for the Cross Canada Lecture Tour. The Reserve Fund was "capped" at $20,000, and any additional amounts are allocated to support the general activities of the Foundation.

The Foundation has also received loans or donations from some of the Local Sections of the CGS and the Local Organizing Committees of the annual CGS Conferences.

In 2007, the Foundation launched its CFG-FCG Website ***www.cfg-fcg.ca*** and established the Canadian Foundation for Geotechnique **National Graduate Scholarship**. This scholarship, with a value of $5,000, is awarded annually to a deserving student entering into, or continuing in, a Masters or PhD program at a Canadian University in any identified field of geotechnique. Also in 2007, the Foundation increased the honorarium associated with the Canadian Geotechnical Colloquium to $5,000.

In April of 2014, the National Graduate Scholarship was renamed as the **Michael Bozozuk National Graduate Scholarship** to honour and to recognize the 43 years of service, dedication and passion that Dr. **Michael Bozozuk** provided to the Foundation, and to the overall geotechnical community in Canada. The inaugural presentation of this scholarship was made in October 2014 at the annual CGS Conference in Regina, SK.

In the fall of 2008, the Foundation established a **Legacy Donor** program to honour individuals who have donated, or who donate, more than $25,000 to the Foundation. To date our Legacy Donors are Dr. **Jack Mollard**, Mr. **Charlie Ripley** and Dr. **Ben Torchinsky**.

In the spring of 2011, the Foundation established a **Legacy Corporate Sponsor** program similar to the Legacy Donor program. The Legacy Corporate Sponsor program honours those corporations that have contributed $30,000 or more to the Foundation to fund the Cross Canada Lecture Tours. Currently, Legacy Corporate Sponsors are **AMEC Environment and Infrastructure**, **Stantec/Jacques Whitford**, **Reinforced Earth (Canada)**, **Tetra Tech EBA**, **BGC Engineering** and **Golder Associates Ltd.**

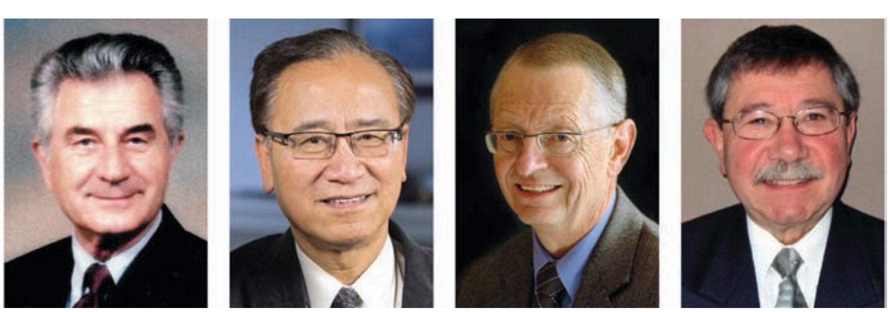
The photo below shows the four presidents of the Foundation from its inception in 2000. A number of other individuals have served in various positions of the Foundation's Board of Directors, including Ms. **E.S. Partsis**, Mr. **M. Bleakney**, Mr. **D. Harding**, Mr. **H. Baker**, and Dr. **R. Benson**.

The current Board of Directors consists of **Dennis Becker** (President), **Kevin Biggar** (Vice-President), **Harry Oussoren** (Treasurer), **Sai Vanapalli** (Secretary), and **Heinrich Heinz** and **Ryan Phillips** as Members at Large. Other Members (formerly Trustees) include **Robert Chapuis**, **Jean Hutchinson**, **Suzanne Lacasse Høeg**, **Jorn Landva**, **Bob Patrick**, **Lynden Penner**, **Siva Sivathayalan**, **Brian Taylor**, **Jean-Pierre Tournier**, and **Gerry Webb**.

Acknowledgements

The Foundation is deeply grateful to the individuals, organizations and corporations associated with the Canadian geotechnical community for their generous support, which in turn promotes excellence in Canadian geotechnique. To learn more about the Foundation and how you can contribute, please visit our website ***http://www.cfg-fcg.ca***

This is the second article of a two-part series based on an article prepared by Dr. Michael Bozozuk and originally published in Geotechnical News in December, 2007. The original article was edited by Drs. Heinrich Heinz and Dennis Becker to fit CGS News publication requirements.



Presidents of the Canadian Foundation for Geotechnique: Dr. M. Bozozuk (2000–2004), Dr. K.T.Law (2004-2008), Mr. D.VanDine (2008-2013), Dr. D. Becker (2014-Present).

Editor

Don Lewycky, P.Eng.
Director of Engineering Services,
City of Edmonton
11004 – 190 Street NW
Edmonton, AB T5S 0G9
Tel.: 780-496-6773
Fax: 780-944-7653
Email: don.lewycky@edmonton.ca

GEOVANCOUVER 2016

69th Annual Canadian Geotechnical Conference October 2nd - 5th, 2016, Vancouver, BC

The Canadian Geotechnical Society (CGS), in collaboration with the Vancouver Geotechnical Society (VGS), invite you to attend the 69th Annual Geotechnical Conference, GeoVancouver 2016 Conference.

The theme of the Conference is **"History and Innovation"**, recognizing the historical achievements and lessons learned over time while highlighting innovation in geotechnical engineering.

PROGRAM HIGHLIGHTS

The Conference will cover a wide range of topics with special sessions that are of local and national relevance to the field of geotechnical engineering.

In addition to the technical program and plenary sessions from renowned keynote speakers, the Conference will include

- Short courses
- Technical tours
- Partners' Activities
- Exhibits
- Networking opportunities at various social events

Visit our website **www.geovancouver2016.com** to learn more about the conference.
Be sure to register before July 31, 2016 to take advantage of the Early Bird rates!

Technical Themes

- Fundamentals
- Case Histories
- Infrastructure Design and Operation
- Geohazards
- Problematic Soils and Ground Improvement
- Soil and Terrain Characterization
- Foundation Design
- Energy Resources
- Cold Regions Engineering
- Geo-Environmental Engineering
- Groundwater and Hydrogeology
- Education and Professional Practice

KEY DATES

JULY 22, 2016
Deadline for full paper submissions

JULY 31, 2016
End of early bird registrations

OCTOBER 2, 2016
Ice Breaker reception

BGC · Clifton Associates · ConeTec · DST consulting engineers · GeoPac · GEOPIER Tensar · Golder Associates · MobileAugers and Research Ltd · Klohn Crippen Berger 65 years of service · MAJOR Partners on the Ground · NILEX Civil Environmental Group · Reinforced Earth · Stantec · Thurber Engineering Ltd. · WSP

Introduction by John Dunnicliff, Editor

This is the 86th episode of GIN. Three articles this time.

People issues

I've often maintained that what I call "people issues" frequently overshadow the importance of technical issues. The article by Martin Beth of Soldata Group presents eight common sense rules for successful monitoring, all of which relate to people issues. In my view this is **MUST READING** by all who have a stake in our goal of obtaining high quality and relevant data.

More on vibration monitoring

The previous GIN included an article by Bob Turnbull of Instantel titled "The fundamentals of vibration monitoring – things to consider". As a follow-up to this, here's an article by Vincent Le Borgne of GKM Consultants titled "Lessons learned in vibration monitoring". The article presents three case histories and conveys yet again that people issues can often overshadow technical issues.

General role of instrumentation, and summaries of instruments that can be considered for helping to provide answers to possible geotechnical questions.

The previous two GINs included articles about instrumentation for braced excavations and embankments on soft ground. Here's one about cut slopes and landslides.

Call for author(s) for one or more articles on monitoring embankment dams

I'd like to publish something similar to the above three article for embankment dams, but am not competent to write it, so I'm looking for a possible author or authors. Some suggestions for content are given below.

It seems to me that the article should have some or all of the following content:

- Monitoring existing embankment dams where there is no evidence of a problem
- Monitoring existing embankment dams where there is evidence of a problem
- Monitoring new embankment dams
- Potential failure mode analysis

It also seems to me that, in contrast to other articles in this series, the types of instruments to be considered for helping to provide answers to the various geotechnical questions are too numerous to be included, but perhaps some general guidance can be given.

Any takers?

Closure

Please send an abstract of an article for GIN to john@dunnicliff.eclipse.co.uk—see the guidelines on www.geotechnicalnews.com/instrumentation_news.php

Yung sing ("drink and win") – China. This from a website with toasts. We lived in Hong Kong for several years in the 1960s and became (very!) familiar with the toast "Yam sing", which we understood to mean "knock it all back without stopping". Does anybody know whether "Yung sing" means the same?

Eight common sense rules for successful monitoring

Martin Beth

Introduction

Geotechnical, structural and environmental monitoring is becoming a standard requirement on civil engineering construction and mining projects, and the amount of recorded data increases rapidly. It is very important to understand that if the monitored data are not of sufficient quality, or if the data are just data instead of useful information to help reduce risks and enhance operations, then monitoring is a waste of money.

The writer would like, though this article, to help convince decision makers to make the right choices when dealing with monitoring.

This article is based on "The challenges of supplying good quality and useful data for significant projects", presented at the Symposium on Field Measurements in GeoMechanics (FMGM), Sydney 2015.

Monitoring program and specifications

Rule number 1: The monitoring program must be designed specifically for the project, and justified by the project needs. (See Figure 1)**.**

The key point is to understand the geotechnical and structural behaviour of the site. Each instrument or group of instruments must be aimed at answering at least one specific question, or one specific problem. A common mistake is when the monitoring design of a past project is copied, totally or in part, to other projects. Consequences will be either:

1. Under design: For example, weekly manual survey is still found in some specifications, in cases where the risks and the potential onset of occurrence of the risks would suggest that hourly readings would be more appropriate. As a consequence the risks on site are not adequately covered, giving rise to potential incident or accident. Or
2. Over design: Contrary to what one might think, this is seen nearly as often as under design. In such cases, instruments are installed that were not really needed. The end result is that the site stakeholders will view the monitoring as an unnecessary expense, and not as it should be - a risk minimisation tool.

Rule number 2: Specifications must be clear, listing clear objectives including accuracy (see note at the bottom of the page about the word accuracy), and leaving some degrees of liberty regarding the methods to be used.

The major considerations that can help increase the quality of the measurements are as follows:

1. List clear objectives, if possible listing the engineering values to be obtained, the frequency and the required accuracy.
2. If possible, liberty should be given to the monitoring contractors to select the monitoring system they will use to answer these objectives. The designers/specifiers cannot be experts in all the techniques that they may specify. By leaving some liberty to the monitoring contractors the best value for money will be achieved.
3. The required accuracy of instruments should be achievable on-site – this is **not** the same thing as accuracy determined in the laboratory. If possible, the definition of the accuracy should be detailed in the specification, and also the way that it can be measured. Some liberty can be taken with the official international vocabulary of metrology in order to define something that can be estimated. For example, the accuracy might be defined as "the band containing 80% of the values during 12 consecutive hours with no work and temperature variation of less than 10°C". A full article could be written on this subject!
4. The required accuracy should be at a level that is necessary and reasonable. Do not over-specify here, as those monitoring contractors who wish to comply with the specifications will see increased cost, and those who disregard the specifications will end up winning the job. Sometimes we see requirements for +/-0.1 mm accuracy when 0.5 mm or 1 mm would be sufficient.

Figure 1. Unsuitable design, under design and over design.

Note. The word "accuracy" is used throughout the text. Depending on the instruments used, "accuracy" is correct, but for some instruments a more appropriate word is "precision" or "accuracy of change".

5. Clarify how the specified requirements will be enforced, and specify clear financial penalties in case of non-compliance. Be aware that a specification for uninterrupted measurements with less than four hours downtime for repairs will lead to high service cost for those who respect the specifications. Indeed, the monitoring contractor will need to have one or more highly trained specialist(s), equipped with all repair and replacement equipment, paid on-call, and probably housed in close vicinity to the project.

Monitoring budget and procurement

Rule number 3: Ensure that there is an adequate monitoring budget. Allocating an insufficient budget might end up in wasted money.

Often given insufficient attention, sometimes forgotten, the budget allocated to the monitoring will have a major influence on the quality and usefulness of the data that will be obtained. For geotechnical construction a general rule of thumb is that 1% to 2% of the construction budget is generally adequate for a comprehensive monitoring program. Of course this in only a general idea as, following rule number 1, the extent of monitoring depends on the project needs, in particular the degree of risk. On a site with no risks the budget can be zero, on a site with complex issues the budget could be 4% or more. With proper monitoring put in place, risks can be significantly reduced, therefore potentially saving huge costs. Alternately, if the monitoring budget is too low, the data provided may be of such bad quality that it will prove unusable, and whatever small amount was spent on the monitoring will be wasted money.

Rule number 4: No low-bid procurement for services of the monitoring contractor

Selecting the monitoring contractor based on low-bid is not recommended. In North America the practice is very much state/province dependent, but in most cases the low-bid method is selected, whether in public or private tenders.

In Europe the technical proposal is now considered carefully in public tenders, and acceptance is regularly given to the best proposal after an analysis of both cost and technical issues. However in private tenders, i.e. when the monitoring contractor is selected by the construction contractor, then in most case the low-bid will be chosen.

This brings further case to the defenders of the fact that the monitoring contract is better placed directly with the owner, rather than through a construction contractor. This subject has already been much discussed in previous GIN issues.

One could argue that it is up to monitoring contractors to avoid low bidding. It is a complex decision to decide on the financial limit below which it is better not to do the job. But accepting a contract below that financial limit will result in not being able to provide quality data, thus putting both the job and the company's reputation at risk.

Project management

Rule number 5: Provide strong enforcement of the specifications.

It is important for the owner and the project designer to ensure they will have the power to demand high quality data during the project duration. It is not as trivial as it may appear to enforce, during the contract, what was stated in the specifications: the pressure of the day-to-day site activities, the complexity of leveraging on a contractor or, even more complicated, a construction contractor's subcontractor, all lend themselves towards cutting corners and taking liberties with the specifications. Financial penalties are a possible way to maintain this pressure. This is only achievable if the specifications state clearly the rules, enforcement and verification of those rules.

Monitoring contractor

Rule number 6: Ensure that the monitoring contractor's team is experienced and focused on data quality.

Even with modern day automatic instruments, the final quality of the monitoring relies mainly on the quality of the monitoring contractor's team on site and off site.

The project manager on a large monitoring site acts as the leader for the whole team. The project manager is in a difficult position in that he is also the guarantor, on behalf of his company, of the financial success of the project. A good project manager will understand the necessary balance between financial and technical success. The search for data quality must be at the forefront of the whole company and hierarchy to ensure the proper decisions are taken, even in difficult times.

Figure 2. Team work and understanding what we measure. Credit: Comet Photoshopping / Dieter Enz

The whole team should be trained regularly to be able to perform tasks in an optimum manner. Many monitoring tasks appear simple at first, but can easily lead to false results when not carried out properly. At least one engineer, not necessarily the project manager, should be the quality "control tower", capable of solving any specific technical difficulties, and training the team to check their readings and to detect their own mistakes. It is desirable to have a good proportion of the monitoring team, and especially those in direct contact with the owner and

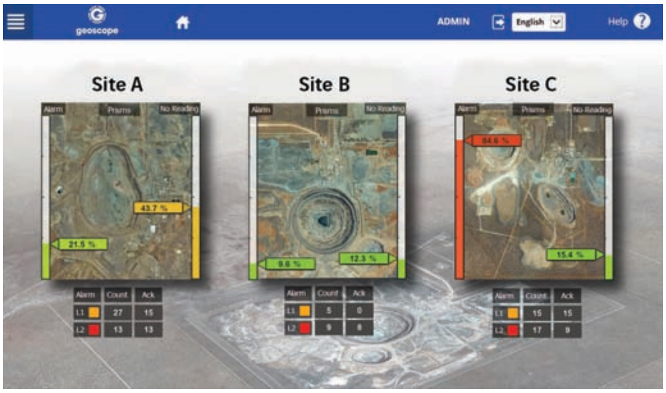


Figure 3. Weather map approach: "board" showing a summary of three sites on one page.

the project designer, coming from a geotechnical or civil engineering background. The quality of the monitoring service is significantly increased when what is being measured is understood. See Figure 2.

QA/QC

Rule number 7: Ensure quality control on the measurements, and actively maintain the monitoring systems.

First, instruments must be installed properly. For many instruments, poor quality of installation will render future measurements impossible or will deliver very poor quality data. Ideally "final control sheets" are put in place, which list, for each instrument, the quality control to be carried out. When possible the control consists of applying a known variation to what is being measured, and checking on the final output (the report or the monitoring database screen) whether the variation is correct. It is surprising how many mistakes can be detected using this method. Typically, these include factors of 10 or 1000, inverted axes, etc.

After installation comes the monitoring. Many clients question the reason for having the expense of data managers and data control on site. A common comment is "the instruments are automatic, so you do not need anybody on site". But without continuous quality control, the systems, whether manual or automatic, will quickly drift. Such control can be automatic though data analysis algorithms, but human brain power is also necessary. Data managers analyse the alarms and conduct corrective actions if necessary, they check the manually acquired data, and they are in charge of carrying out detailed quality checks on selected instruments.

Finally, depending on the accepted level of risk, sufficient spares parts and redundancy must be provided and included in the budget.

Data to information

Rule number 8: Include added value tools to maximize the use of the monitoring data.

The primary deliverable of any monitoring system are valid measurement data. This is a major achievement in itself. But the next question to address is: how can the usefulness of the monitoring data be maximised for the users, considering that data are useless if not understood?

With this objective in mind, all the following are important features:

- Data integration (all data, from all sources, in a single system)
- Data fusion (cross correlation of information from different sources)
- Alarm velocity and data velocity (rapid delivery of the alarm and rapid analysis of its causes)
- Alarm management (acknowledgement, by whom, why, etc.),
- Weather map approach or dashboard. This is the ability to display in a very simple and effective way a huge volume of more or less complex data, so one can understand what's going on at a glance, in a similar way to a meteo map on your TV screen summarizing the calculations of some of the biggest computers on earth. See Figure 3 for example, where three sites are summarized on one page, showing for each site the number and percentage of sensors not reading (for example disconnected if automatic, or the planned frequency is not respected if manual), the number and percentage of alarms of type 1 and type 2 (count L1 and count L2), and the number of alarms that have been acknowledged (i.e. controlled and commented by an operator). The colors of the squares and side bars help to understand at a glance the status at the monitoring site.
- Journal (the monitoring system records a journal of internal or external events that is presented alongside the data to help the analysis)

Conclusion

The factors influencing the success, partial failure, or total failure of a monitoring project are numerous, starting from the design phase, through to procurement of the monitoring contractor, installation of the instruments, to data collection, pre-analysis, data presentation and reporting.

Martin Beth,
Soldata Group
3120 Route d'Avignon,
13 090 Aix en Provence
France.
Tel +(33) 4 42 21 72 11
E-mail:martin.beth@soldatagroup.com

Lessons learned in vibration monitoring

Vincent Le Borgne

Introduction

Vibration monitoring is growing in popularity as a complement to geotechnical monitoring because infrastructure work generates noise and vibration that can have deleterious effects on structures and people. To ensure compliance with local ordinances and to protect sensitive structures, long-term vibration monitoring is more and more commonly used. The relevance of vibration monitoring was recently brought to the attention of the readers of Geotechnical Instrumentation News (GIN) by Turnbull in a March 2016 article entitled "The fundamentals of vibration monitoring - things to consider". The article provides an overview of the technical requirements of vibration monitoring.

Our company has worked on several major projects in which vibration monitoring was a key component in addition to "traditional" geotechnical monitoring. In each of the projects detailed in this article, the first and perhaps most important thing to be decided was the goal of vibration monitoring. These goals led to the choice of the acceptable vibration limits and the appropriate sensors and data loggers. Finally, the method of data collection was determined according to the requirements of the client and the technological limitations of the equipment used. In addition to giving examples for each of the steps, we will explain how, despite following this basic methodology, unforeseen issues and human elements end up playing key parts in the lessons learned in vibration monitoring.

Figure 1. The geophone system and the historical building to be monitored.

Project 1

Technical requirements

In this project, vibration monitoring was required for the construction of a tunnel linking a water treatment plant and Lake Ontario. Vibration had to be maintained below a certain threshold for several reasons: to ensure the well-being of residents; to protect private buildings and homes; and to protect a historical building that was identified as being more prone to vibration-induced damage. Near the historical building, peak particle velocity (PPV) of 2 mm/s at frequency below 100 Hz was chosen as the threshold not to be crossed. For other buildings, the threshold was 8 mm/s at less than 4 Hz, 15 mm/s between 4 and 10 Hz and 25 mm/s above 10 Hz. The threshold is varied as a function of frequency because low frequency vibration is much more damaging than high frequency vibration for any given PPV. Sensors and loggers were thus chosen according to these requirements.

Stations were installed at eight locations clustered around shafts and close to the historical building. The installation next to the historical building is shown in Figure 1. The assembled system is anchored to the concrete slab, and the old stone and mortar wall behind can clearly be seen. There are whiter parts in the wall where the mortar has been repaired before, showing that this building is indeed weakened and requires extra caution.

To minimize long-term costs to the client, vibration data are uploaded daily, automatically to the client's server, where engineers can access it. This is achieved by hooking up a cellular modem to the logger and setting up scheduled data transfers. This passive method of data retrieval is well-suited for this application since we were confident that the generated vibration from tunnel construction would never exceed the threshold, thus eliminating the need for real-time alarms.

Lessons learned

Despite a smooth start, unforeseen equipment failures forced us to quickly review our setups and devise an action plant to ensure as little data as possible would be lost. Of the failures, the most common one was unreliable cellular modem communications. The modems would hang and generate issues in the transferred data, and create doubts regarding system reliability. There was a very real risk that tunnel construction would go on without our system continuously providing evidence that bylaws and other requirements were being followed. In this context, a well-prepared contingency plan is a necessity to ensure full protection for the client.

Beyond these hiccups, the main lesson learned from this project is not about choice of instruments, installation or data analysis. The main challenge proved to be communicating efficiently with the client. On several occasions, we have gone over with the client how the system works, how to configure it and how to extract data. Despite offering training sessions and providing several training documents, the client still had difficulty maintaining and using the vibration monitoring equipment.

There was a fairly high turnover rate for the people in charge of this equipment, and information would be lost from person to the next. Compounding this issue, the people in charge have often been temporary student workers, which almost guarantees their contract ends before their successor is hired and thus that they had not passed on their knowledge correctly before leaving. In the context of ensuring compliance to the project requirements, it is necessary to plan with the client how knowledge will be transferred from us to them and maintained within their team.

In short, the general outline of vibration monitoring was followed: vibration sources and limits were identified; instruments and measurement locations were chosen accordingly; and the system was set up according to the requirements. The main lesson drawn from this project is that for the system to work as intended, communication with the client and technological transfer are almost as, if not more, important than the technical aspects of the system.

Project 2

Technical requirements

Large cracks running along several hundred meters in a large wastewater sewer compromised security during infrastructure work in the vicinity of the tunnel. A collapse of the sewer could lead to flooding with wastewater in a very densely populated area. Given the length and the width of the cracks (over 5 cm), very stringent vibration criteria were set: vibration should never exceed 2 mm/s for low frequency. Similarly, cracks should not open or close at all during infrastructure work in the vicinity. In consequence, two main types of instruments were used: 12.5 mm-range vibrating-wire crackmeters and geophones.

In both cases, data are retrieved in a trailer where an engineer continuously monitors vibration and crack deformation. The vibration dataloggers are linked to a cellular modem which can transfer data to a server. Special software monitors incoming data and sends out alarm e-mails as needed.

In most projects an alarm e-mail sent out within 15 minutes of vibration exceeding the threshold is considered satisfactory. The major public safety risk that a collapse would cause made

Figure 2. A geophone installed in a wastewater tunnel.

it preferable that an engineer would monitor the data in real-time to make any work stop in under a minute.

Lessons learned

The unique work conditions of a large wastewater sewer pose significant difficulties. Work in sanitary sewers is accompanied by a slew of worker safety rules. Installation of instruments was conducted by workers accustomed to confined spaces who had never installed geotechnical instrumentation. The first step was to prepare a course to teach them how to install the instruments in the tunnel. This was achieved with hands-on demos that had the workers install instruments on a concrete jersey (a modular road barrier) and with preparation of drilling templates with every tool needed properly identified. Despite thorough preparation, we rapidly came to the conclusion that it was necessary to be available during installation should any issue arise. It would be very difficult and costly to fix an improperly mounted or damaged instrument and we made sure to provide whatever help we could through an unreliable radio link. In addition to these considerations, working in a sewer raised logistical issues. Workers wear a special combination with respirators, heavy boots, a rubber dry suit, a radio, and three pairs of gloves that hinder their.

Due to the high water level and flow, protective equipment for the instruments had to be designed. After installation of each geophone, a metal cover was bolted on top to protect it from impacts from smaller debris and to deflect heavy debris carried by water. Geophone casings were also filled with epoxy resin to make them fully waterproof and their cables were fed into a flexible metal conduit that was bolted to the wall. This is illustrated in Figure 2, where a geophone installed inside the tunnel, with the protective cover, the conduit for the cable, and one of the large cracks running alongside are displayed. Similar protection was provided to the crackmeters. This was all done because maintenance would have proven challenging. Access is difficult and restricted, cables ae bolted to the wall and vision and dexterity are severely limited in the tunnel. Flowing water during rainstorms did not significantly affect vibration measurements. Water flow barely registered on the geophones and was not anywhere near the 2 mm/s threshold. Finally, crackmeters showed that the cracks expand and contract as the tunnel heats up and cools down. The main goal of this project was to ensure that the tunnel would remain stable during construction work. It did remain stable and no crack opening or contraction were observed beyond thermal effects.

Project 2 brought up a plethora of challenges that needed very careful planning. In this project, as a follow up to project 1, we have seen the value of putting a deliberate effort into communications with the client from the very beginning of the planning stages. Doing so ensured rapid and correct installation of the instruments. To sum up, conducting a successful vibration monitoring project goes beyond simple technical considerations.

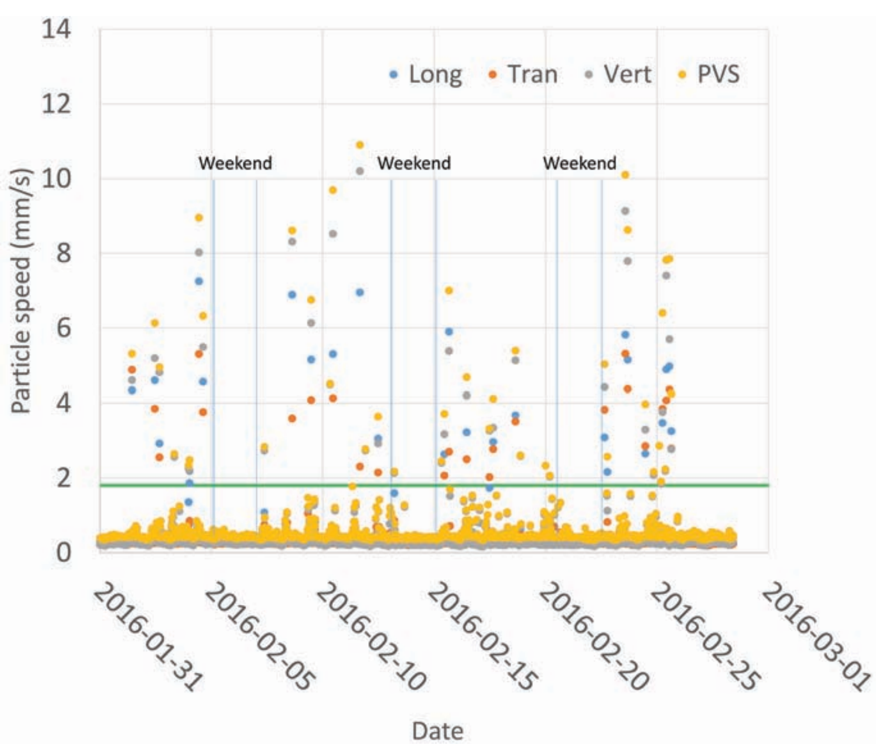


Figure 3. One-month sample of vibration measurements near a shaft.

Project 3

Technical requirements

The last project is a new 5 km long sewer tunnel being constructed underneath a densely populated area. Similar to project 1, vibration had to be monitored around the shafts and along the tunnel route. In addition to vibration monitoring, "traditional" geotechnical instruments were installed (inclinometers and multipoint borehole extensometers) to measure the effects of tunneling and to ensure that no convergence or settlement would threaten the surrounding structures. Lastly, noise monitoring was also undertaken to ensure compliance with bylaws concerning noise emissions.

Lessons learned

It was estimated that the blasting schedule would pose almost no risk of damaging buildings. Indeed, 25 mm/s is the accepted threshold for modern buildings and the blasting schedule was designed to keep vibration much lower for any single event. Monitoring was thus mostly meant to reassure residents, because humans feel vibration up to ten times less intense than those that normally pose a threat to buildings. Having this system in place also ensured that if any blasting event was higher than expected, it would be quantified and any resulting damage could be assessed subsequently.

With event-based and general monitoring of blasting in mind, an automated data collection system with cellular modems was put in place to ensure that data were transmitted rapidly to the server. Specifications required that the blasting foreman must be alerted within 15 minutes by the construction contractor if any vibration crossed the threshold. To this end, the specifications written by the city engineers required alarms to be sent out to the construction contractor upon 2.5 mm/s peak vector sum (PVS) for any frequency. This type of arrangement is fairly common to ensure that work cannot continue while generating harmful levels of vibration.

Peak vector sum is defined by the following equation:

$$PVS = \sqrt{tran^2 + vert^2 + long^2} \quad (1)$$

in which *tran*, *vert* and *long* are respectively the transverse, vertical and longitudinal PPV. However, the datalogger could only relay alarms on the PPV and not on the PVS. This raises the issue that each axis could be below 2.5 mm/s PPV while their PVS is above 2.5 mm/s, and no alarm e-mail would be sent out. As a compromise, alarms are relayed if any one of the axes are above 1.8 mm/s, which leads to a maximal possible peak vector sum of 3.11 mm/s according to equation (1). Lower values could have led to too many false positives and hampered progress of the tunnel construction. Figure 3 shows the measured *tran*, *vert*, *long* and PVS values over a one-month period. The green line at 1.8 mm/s shows the alarm threshold. It can be seen that the measured vibration are typically much lower than the 1.8 mm/s threshold, blasting events have created PPV as high as 11 mm/s. There are also clear lulls during weekends where little to no vibration is measured.

The automated system was required and expected by the client to be functioning twenty four hours per day. Clients and construction contractors expect this to be a cheap and straightforward affair that requires little to no maintenance. However, the large number of components (batteries, casing, logger, sensors, and cellular modems) make these goals difficult to reach. The loggers and cellular modems are finicky and sometimes unreliable, occasionally requiring to be reset on-site. Having staff available to check on the systems weekly and to replace batteries and recharge units, made vibration monitoring much more involved than originally planned.

This project proved to be fairly straightforward once the technical issues were settled. A lesson to be drawn from this project is that, vibration criteria can be chosen for their effects on residents rather than only to protect buildings and infrastructure, and systems were designed to provide automated alarm e-mails.

Conclusions

In every vibration monitoring project, technical requirements come first: frequency range, sensitivity, measurement range, etc. Choosing thresholds according to the specific needs is possibly the most critical decision for this type of monitoring. Other important considerations include that humans are much more sensitive to vibration than structures and that there can be older, more sensitive structures. However, creating a good monitoring project that fulfills its duty also requires deliberate planning and communication with the client, from the planning phase to its final execution. This is an often overlooked point that proves to be very important in vibration monitoring, perhaps even more so than in "traditional" geotechnical monitoring because it is chiefly implemented for safety, legal and wellbeing reasons.

Vincent Le Borgne
GKM Consultants, 1430 Hocquart,
Saint-Bruno, J3V6E1, Canada
Tel. (450) 441-5444 (ext 207)
E: vleborgne@gkmconsultants.com

General role of instrumentation, and summaries of instruments that can be considered for helping to provide answers to possible geotechnical questions. Part 3.

John Dunnicliff

Introduction

This is the third in a series of articles that attempt to identify:

- The general role of instrumentation for various project types.
- The possible geotechnical questions that may arise during design or construction, and that lead to the use of instrumentation
- Some instruments that can be considered for helping to provide answers to those questions.

Part 1, covering internally and externally braced excavations, was in December 2015 GIN.

Part 2, in March 2016 GIN, covered embankments on soft ground. This Part 3 is about cut slopes and landslides in soil and in rock.

The following points were made in the introduction to Part 1, and also apply here:

- Of course it is recognized that there may be additional geotechnical questions and also additional instruments that are not described in this article.
- The sequence of geotechnical questions is intended to match the time sequence in which the question may be addressed during the design, construction, and performance process, and does not indicate any rating of importance.
- The suggestions for types of instruments is not intended to be dogmatic, because the selection always depends on issues specific to each project, and is influenced by the personal experience of the person making the selection. In the tables some of the most likely instruments that can be considered are listed, with other possible types in parentheses.
- The tables include the term "remote methods" for monitoring displacement. An overview of these remote methods is given in a December 2012 GIN article by Paolo Mazzanti (www.geotechnicalnews.com/instrumentation_news.php). Readers who want to learn more about these methods may want to consider participating in the annual International Course on Geotechnical and Structural Monitoring held in Italy (www.geotechnicalmonitoring.com), where they are discussed in detail.

Cut slopes in soil

General role of instrumentation

It is imperative that, prior to planning an instrumentation programme for a cut slope in soil, an engineer first develop one or more working hypotheses for a potential behaviour mechanism. The hypotheses must be based on a comprehensive knowledge of the locations and properties of stratigraphic discontinuities.

Instrumentation can be used to define the groundwater regime prior to excavating a slope. Results of measurements during excavation can be used as a basis for modification of the designed slope angle. Measurements of ground movement and positive or negative groundwater pressure can assist in documenting whether or not performance during and after excavation is in accordance with predicted behaviour. Measurements can also be used to document whether short- and long-term surface and/or subsurface drainage measures are performing effectively. If evidence of instability appears during or after construction, instrumentation plays a role in defining the characteristics of the instability, thus permitting selection of an appropriate remedy.

A very important subset is the case of a cut slope in clay. Here negative pore water pressures generated during excavation can give rise to temporary stability, the lifetime of which will be related to the height of the slope and the slope angle. Therefore monitoring the negative pore water pressures is an effective way of assessing the stability of a cut slope in clay. In some instances the stability may be maintained for long enough to undertake temporary works within the excavation and thereby save on expensive stabilisation measures.

Summary of instruments that can be considered for helping to provide answers to possible geotechnical questions

Table 4 lists the possible geotechnical questions that may lead to the use of instrumentation for cut slopes in soil, together with possible instruments that can be considered for helping to provide answers to those questions.

Landslides in soil

General role of instrumentation

If there is evidence of slope instability, its characteristics must be defined so that any necessary remedial measures may be taken. The question *how much ground is moving?* can be answered by use of instrumentation. The question *why is the ground moving?* will not be answered by instrumentation alone: the answer of course also requires a

Table 4. Some instruments that can be considered for monitoring cut slopes in soil

Possible geotechnical questions	Measurement	Some instruments that can be considered
What are the initial site conditions?	Pore water pressure	Open standpipe piezometers Vibrating wire piezometers installed by the fully-grouted method Flushable piezometers (Pneumatic piezometers)
	Surface displacement	Conventional surveying methods Remote methods (Tiltmeters) (Fiber-optic instruments)
	Subsurface displacement	Inclinometers In-place inclinometers (Time domain reflectometry) (Fiber-optic instruments)
Is the slope stable during excavation?	Surface displacement	Conventional surveying methods Remote methods (Tiltmeters) (Time domain reflectometry) (Fiber-optic instruments)
	Subsurface displacement	Inclinometers In-place inclinometers (Time domain reflectometry) (Fiber-optic instruments)
	Pore water pressure	Vibrating wire piezometers installed by the fully-grouted method Flushable piezometers
Is the slope stable in the long term?	As for “Is the slope stable during excavation?”	As for “Is the slope stable during excavation?”
	Rainfall, for possible correlation with any displacement	Rain gauges
	Load in tiebacks	Load cells

Table 5. Some instruments that can be considered for monitoring landslides in soil

Possible geotechnical questions	Measurement	Some instruments that can be considered
What are the post-landslide conditions?	Pore water pressure	Open standpipe piezometers Vibrating wire piezometers installed by the fully-grouted method Flushable piezometers (Pneumatic piezometers)
	Surface displacment	Conventional surveying methods Remote methods (Tiltmeters) (Fiber-optic instruments)
	Subsurface displacement	Inclinometers In-place inclinometers (Time domain reflectometry) (Fiber-optic instruments)
Is the slope stable in the long term?	As for "What are the post-landslide conditions?"	As for "What are the post-landslide conditions?"
	Rainfall, for possible correlation with any displacement	Rain gauges
	Load in tiebacks	Load cells

complete geotechnical investigation and analysis. Instrumentation also plays a role in monitoring the long-term stability of the slope after remedial measures have been taken.

Summary of instruments that can be considered for helping to provide answers to possible geotechnical questions

Table 5 lists the possible geotechnical questions that may lead to the use of instrumentation for landslides in soil, together with possible instruments that can be considered for helping to provide answers to those questions.

Cut slopes in rock

General role of instrumentation

The general role of instrumentation is identical to the role for cut slopes in soil, as discussed above. However, when planning to monitor the stability of rock slopes, it is important to recognize that if the slope is subject to a brittle failure mode, movement will be sudden. In such cases, geotechnical instrumentation may not be appropriate to forewarn of instability. It may be more appropriate to develop an area-wide correlation between rainfall intensity and slope instability, and to use rainfall measurements to warn of potential problems.

Summary of instruments that can be considered for helping to provide answers to possible geotechnical questions

Table 6 lists the possible geotechnical questions that may lead to the use of instrumentation for cut slopes in rock, together with possible instruments that can be considered for helping to provide answers to those questions.

Landslides in rock

General role of instrumentation

The general role of instrumentation is identical to the role for landslides in soil, as discussed above.

Summary of instruments that can be considered for helping to provide answers to possible geotechnical questions

Table 7 lists the possible geotechnical questions that may lead to the use of instrumentation for landslides in rock, together with possible instruments that can be considered for helping to provide answers to those questions.

Table 6. Some instruments that can be considered for monitoring cut slopes in rock

Possible geotechnical questions	Measurement	Some instruments that can be considered
What are the initial site conditions?	Joint water pressure	Open standpipe piezometers Vibrating wire piezometers installed by the fully-grouted method (Pneumatic piezometers)
	Surface displacement	Conventional surveying methods Remote methods Crack gauges (Tiltmeters) (Fiber-optic instruments)
	Subsurface displacement	Fixed borehole extensometers In-place inclinometers (Acoustic emission monitoring) (Time domain reflectometry) (Fiber-optic instruments)
Is the slope stable during excavation?	Surface displacement	Conventional surveying methods Remote methods Crack gauges (Tiltmeters) (Time domain reflectometry) (Fiber-optic instruments)
	Subsurface displacement	Fixed borehole extensometers In-place inclinometers (Acoustic emission monitoring) (Time domain reflectometry) (Fiber-optic instruments)
	Joint water pressure	Vibrating wire piezometers installed by the fully-grouted method
Is the slope stable in the long term?	As for "Is the slope stable during excavation?"	As for "Is the slope stable during excavation?"
	Rainfall, for possible correlation with any displacement	Rain gauges
	Load in tiebacks	Load cells

Table 7. Some instruments that can be considered for monitoring landslides in rock		
Possible geotechnical questions	**Measurement**	**Some instruments that can be considered**
What are the post-landslide conditions?	As in Table 6 for "What are the initial site conditions?"	As in Table 6 for "What are the initial site conditions?"
Is the slope stable in the long term?	As in Table 6 for "Is the slope stable in the long term?"	As in Table 6 for "Is the slope stable in the long term?"
	Rainfall, for possible correlation with any displacement	Rain gauges
	Load in tiebacks	Load cell

Case History V

extract from Suit is a Four-letter Word

(Hugh Nasmith, 1986)

This case history illustrates the hazard of filling a report with an excess of detail and comments.

A major high-rise office building with several levels of underground parking was planned for an urban development. A geotechnical firm was employed to carry out and report on subsurface conditions. Test drilling established bedrock (a horizontally bedded sedimentary rock) at a shallow depth and the borings were extended to the full depth of the proposed excavation. A professor of geology was retained by the geotechnical consultant and asked to examine and describe the core. His report was very thorough and comprehensive. The age, lithology, structural discontinuities, mineralogy, jointing, bedding, and fossils were described and discussed in detail, even though the report was based on an examination of discontinuous small diameter core. The entire geological description was incorporated in the geotechnical report which became part of the contract documents.

In the course of drilling and blasting the bedrock to excavate for the basement and footings, considerable overbreak occurred which the contractor was obliged to backfill with lean concrete. Blasting was carefully controlled by an explosives expert hence the overbreak could not be attributed to poor procedures.

The contractor claimed for an extra as a result of the overbreak and his "expert" claimed that the contractor relied on the geological description of the core as thinly "bedded" and accordingly made little allowance for overbreak. Photographs taken during construction showed horizontal beds 1.0 to 1.5 meters thick which in terms of mass rock would not be regarded as thinly bedded.

It was concluded that the contractor had a valid claim and was paid an extra. This claim might not have been allowed if the report had merely reported the rock type, elevation and percentage of core recovery (RQD) and included representative photographs of the core.

From a practical point of view the small diameter of the core made it unsuitable for determining the spacing of the bedding planes. It is unlikely that the geologist or the geotechnical engineer anticipated that the rock description would be used to predict the behaviour of the rock when excavated. If the professor had realized the importance that would be attached to the term "thinly bedded" he would no doubt have considered the use of the term more carefully. An examination of a nearby rock outcrop would probably have been more informative than the core fragments. A cynic might suspect that the contractor only studied the description of the rock in detail when he realized the extent and cost of the overbreak.

When you pad out a report with a mass of extraneous detail and comments, you are providing answers to questions which have not been asked and the answers you have given may well be wrong or misleading.

Paolo Gazzarrini

Overture

43rd episode of the Grout Line and for this issue the third and final part of the article prepared by by Clif Kettle, Technical Manager, Bachy Soletanche Ltd., Burscough, Lancashire, UK,(clif.kettle@bacsol.co.uk) and Maren Katterbach, Project Engineer, Lombardi Engineering Ltd., Minusio, Switzerland (maren.katterbach@lombardi.ch).

Before the article, an "errata corrige" related to the March 2016 issue

As usual, I make the same request, asking you to send me your grouting comments or grouting stories or case histories. My coordinates remain:

Paolo Gazzarrini, paolo@paologaz.com, paologaz@shaw.ca or paolo@groutline.com.

Ciao! Cheers!

ERRATA CORRIGE of the March 2016 Grout Line issue

1) The title of the cover page states: "Seepage evidence at Carno Dam, South Wales". It might not have have not been clear that the seepage had occurred BEFORE the grouting treatment. After the grouting treatment the seepage disappeared.

2) There was some confusion at pages 45 and 46.

2.1) First of all figure 7 is referring to Carno Dam, and figures 8 to 11 to Lee tunnel.

2.1) In figures 8 to 11 (Lee Tunnel) some captions were incorrect.

Figure 11 should be figure 8 and figure 8 should be figure 9.

Consequently, the correct sequence is: figure 11, 8, 9 and 10.

The complete and corrected article is downloadable at: http://www.groutline.com/Articles/42GroutlineMarch 2016.pdf

GIN Method (part III)

Clif Kettle & Maren Katterbach

Case History 3 - Wimbleball Dam, England, 2003, 2014

Figure 1. Wimbleball Dam foundation excavation.

Wimbleball dam is a lightweight buttress dam which has suffered seepage through the grout curtain, and through the left abutment ever since construction. Seepage had been increasing steadily since the construction in 1979 up to 2003, despite at least two attempts at remedial grouting with classical techniques.

Because of the poor quality of the bedrock a very large excavation had been undertaken during the initial construction of the dam in 1979. The

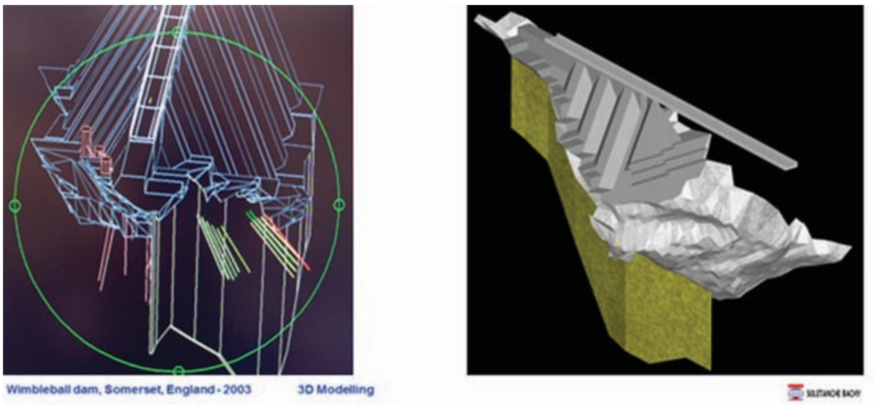


Figure 2. 3D model of Wimbleball Dam foundation based on historic records.

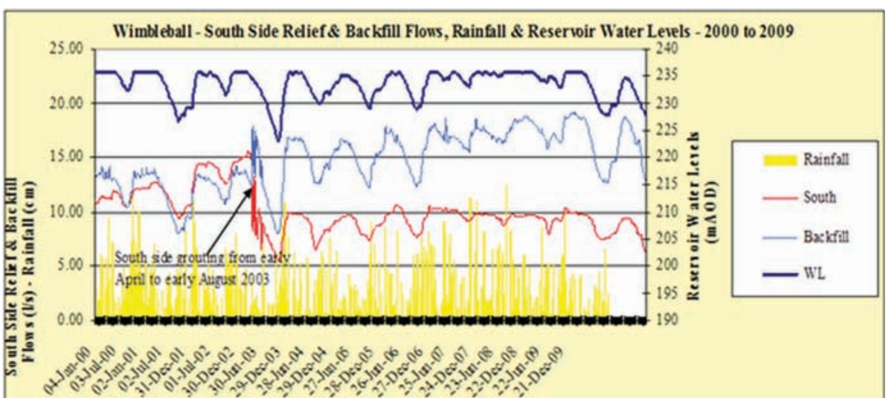


Figure 3. Seepage development before and after the grouting works in 2003.

size and area of the base slab, compared to the relatively light weight the concrete face and buttresses, has led to the dam being extremely sensitive to drainage through the curtain and resulting hydraulic uplift. In addition, many fissures have been identified as having a clay infill, and there has been evidence over the years of progressive washout of this clay infill, evidenced by clay and sand sedimentation within the under drainage system, leading to fears that the foundation seepages may increase in the medium to long-term and undermine dam stability.

The dam abutment and foundations were also significantly affected by the development of significant stress release fractures during construction, and these, combined with the multiple natural joint sets have led to a structurally complex foundation rock which is highly fissured with permeabilities of up to 3,000 Lugeons.

Remedial works, 2003 season

It was fortunate that during the summer of 2003 a highly targeted injection program was carried out to address a preferred seepage path identified via drainage and sedimentation in the under drainage system. This limited program included just 9 no. boreholes, and injection used the C3S stabilised bentonite cement grout for permeation of fine fissures, placed using the GIN technique.

As a precursor to this work a very detailed 3-D model of the dam foundations and the excavation profile was prepared using historic records from the date of the dam construction (see Figure 2). This 3-D model proved to be invaluable both for the 2003 season works and also for the final remediation program which was executed during 2014. This model was used both for the 3-D design of all subsequent borehole geometry, and for the presentation and analysis of the graphical data from the injection works.

The execution of this limited amount of works provided extremely valuable information which was able to provide a basis for the design of the 2014 remedial works. The injection program:

a. identified the presence of voids and major fractures within the immediate dam foundations below the base slab
b. highlighted hydraulic connections over distances of up to 32 m between individual boreholes
c. provided an average grout absorption per linear metre of borehole, and
d. verified the geometry of the rock profile and dam foundation.

Despite a very limited program, the 2003 works achieved three major objectives

a. it reduced total seepages through the relief wells by 44% in the target area, dropping the total flows sufficiently below the critical level to allow the client several years to develop a comprehensive and final solution
b. it arrested completely the progressive increase in erosion within the fissure network, so that the reduced flows stabilised, and remained constant, over the succeeding 11 years
c. it verified that a permanent solution was achievable with stable residual flows, using fluidified mixes and GIN technology

Figure 3 shows the trend of increasing seepage up to the 2003 grouting works, and stable seepage thereafter through the South abutment, with flows diverting around the curtain into the backfill. The graphical plots in Figure 4 indicate high absorptions close to the dam foundations.

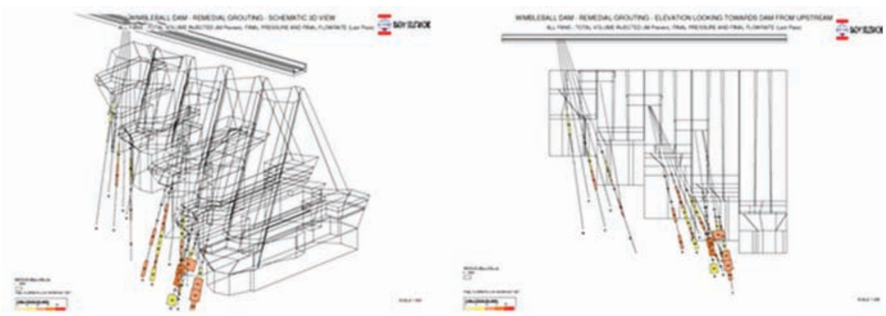


Figure 4. Grout absorptions at the dam foundation.

Remedial works, 2014 season

Although after the 2003 season remedial works the total relief well seepages had been reduced by 44%, there had been no further reduction over the intervening 11 years. The client wanted a permanent solution to reduce the seepages through the grout curtain to a level which would effectively remove the risk of degradation

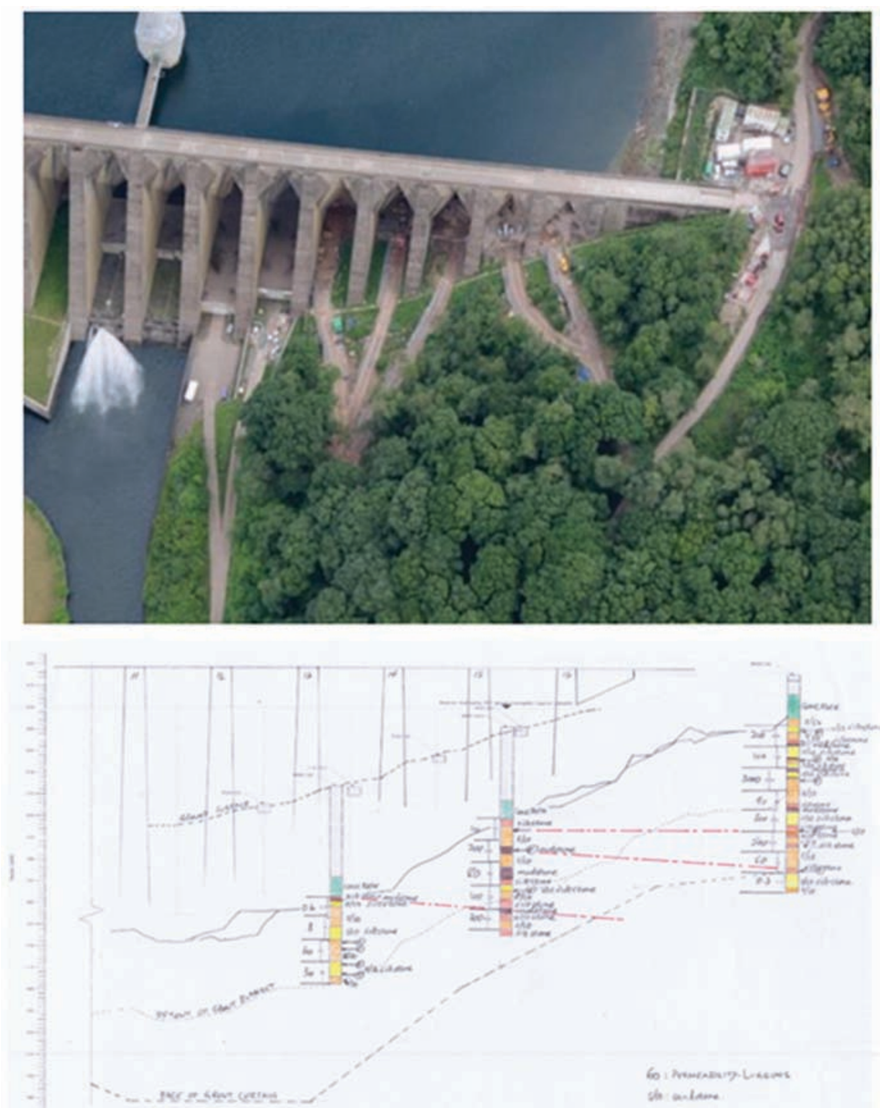


Figure 5. Wimbleball Dam and its foundation stratigraphy.

of the rock formation by the erosion and washout of clay material within fissures. The client's designer decided to construct a new, single row grout curtain, downstream of the original, directly below the base plinth of the dam, and extending over a length of 150 m up to the left abutment.

In order to achieve the core objectives, the designer prepared an extremely demanding specification which required

- a residual permeability of just 0.1 Lugeon units, equivalent to 1 x 10^{-8} m/s.
- a grout mix with reduced cement content to resist the degradation caused by the super soft reservoir water, which had a Langelier index of -2.5.
- a very high level of process control, including the use of computer-piloted pumps.
- the use of GIN grouting to control the works and avoid excessive uplift pressures on the relatively light dam structure
- a full-scale verification trial on site to verify the effectiveness of the selected grout mixes, injection parameters, and curtain geometry

Given the range of permeabilities - from the 0.1 Lu specified residual permeability, up to the 3'000 Lu identified in the extensive pre-grouting investigations, and the difficulty of addressing this range with the single row grout curtain, it was clear that the mix design would be a very key element of the works, to allow for permeation of both very fine fissures, and heavily fractured zones with significant voids and open fissures.

An extensive programme of mix design was carried out within the laboratories of Bachy Soletanche to develop the grout mix(es) necessary to deal with such a very wide range of rock conditions. It was decided to use a slag based cement to address the issue of grout degradation arising from the Langelier index, and over a period

Table 1: Preliminary hydraulic testing results from validation boreholes

Hydraulic Testing
Preliminary results
Borehole VH04

Interval	Depth (1) Top [m]	Depth (1) Bottom [m]	Test Length [m]	Event	P i [kPa]	P END [kPa]	ΔP [kPa]	ΔP [kPa]	Flow Rate [ml/min]	Duration HI [sec]	Duration Steady [sec]	Remarks	Lugeon Units [l/m/min]	h "s" (2) [mbgl]	Steady-state approximation (SSA) T m2/s	Steady-state approximation (SSA) K m/s	Straight-line analysis (transient) (SLA) T m2/s	Straight-line analysis (transient) (SLA) K m/s
VH04-i2	37.80	49.00	11.20	HI	403.63	522.44	118.81	1.19	1.126	1800	1800	Single packer test, bottom of interval is uncertain, the borehole was dipped before the test to 40 mbgl, here the interval is assumed to be 11.2m. HI : straight line fit on late time data, SSA on late time data	0.8	5.00	1.70E-06	1.52E-07	1.43E-06	1.28E-07
VH04-i3	32.80	37.81	5.01	HI	355.34	632.51	277.17		404	1241	1000	HI: little bypass P1(14.8 kPa), straight line fit on middle time data, SSA on late time data	0.29	5.02	1.86E-07	3.71E-08	1.78E-07	3.55E-08
VH04-i4	27.80	32.81	5.01	HI	250.01	636.15	386.14	3.86	59	916	600	HI: very low transmissivity zone, no straight line fit possible, SSA on late time data	0.03	10.86	1.31E-08	2.62E-09	***	***
VH04-i5	25.20	30.21	5.01	HI	151.06	561.44	410.38	4.10	49	1018	200	HI: very low transmissivity zone, no straight line fit possible, SSA on middle to late time data	0.02	18.39	7.70E-09	1.54E-09	***	***
VH04-i6	20.20	25.21	5.01	HI	157.97	498.29	340.32	3.40	44	1073	500	HI: very low transmissivity zone, no straight line fit possible, SSA on late time data	0.03	12.79	1.05E-08	2.10E-09	***	***
VH04-i7	16.00	21.01	5.01	HI	183.41	558.84	375.43	3.75	1127	1127	700	Upper packer is inflated in the concrete, HI: straiht line fit on middle time data, SSA on late time data, positive skin effects SSA results	0.13	6.08	7.04E-08	1.41E-08	1.40E-07	2.79E-08

(1) Depth along borehole axia (2) Head in vertical meters below ground level

P i	Initial pressure, or best estimate of "static" pressure
P END	Pressure at end of test
h s	"static" head, metres below ground level
P1	Pressure below bottom packer
P3	Pressure above top packer
SL	Straight-line
SSA	Steady-state approximation

Assumed spez. Storativity	*[l / min]*	2.00E-06
Well radius	*[m]*	0.048
Sensor P2 above top interval	*[m]*	1.66
P. atm (P2 prior to installation)	*[kPa]*	105.06
Inclination (from Horizontal)	*[^{0}]*	78.7

of weeks of laboratory testing, and full-scale mix testing using the same mixer which would eventually be used to the main works, two grout mix designs were eventually selected.

a. for the primary holes, a slag-based cement using 32 µ material, with the addition of a super-plasticiser, and with a gelling agent to assist in retaining grout in situ despite the significant hydraulic head across the grout curtain. In addition, the mix was tested with an accelerator in case significant voids or fissures were encountered which could lead to the grout being washed away and into the river downstream of the dam.

b. For the secondary and tertiary holes, a slag based cement using 12 µ material, with the addition of a super-plasticiser and gelling agent

In order to avoid premature blockage of the extremely fine fissures, it was necessary that the rheological properties of the grout remain constant, well controlled, and well understood throughout the injection, and considerable care was taken in ensuring that the gel times of the two mixes would allow injection of the full target volume when using the GIN technique. A minimum gel time of not less than 2.5 hours was established.

The GIN value was carefully selected to reflect the average absorption per linear metre experience during the 2003 works, the designers' require-

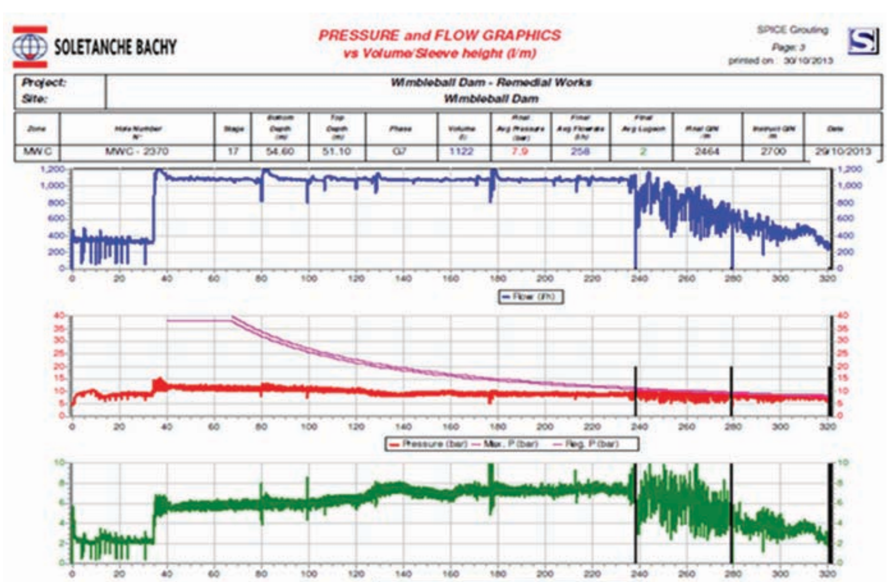


Figure 6. Pressure and flow graphics.

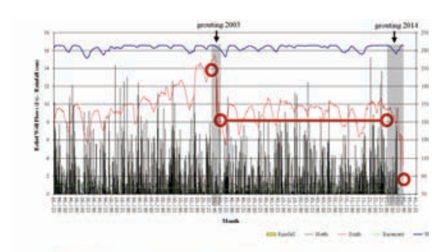


Figure 7. Wimbleball Dam 2013-14: Leakage history showing impact on phase 2 grouting.

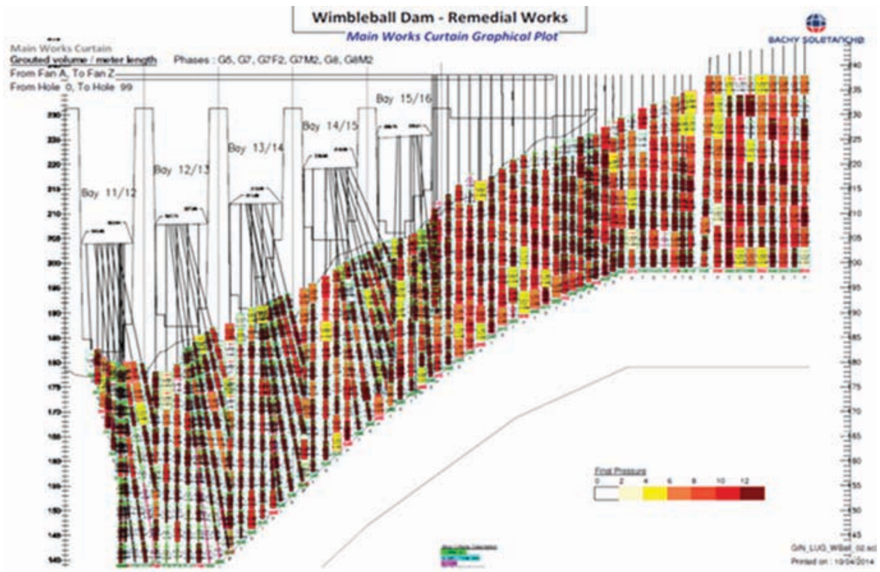


Figure 8. Wimbleball Dam - Graphical plot of final grouting pressures.

Figure 9. Symvoulos Dam on soft limestone bedrock.

ment for a cut-off approaching 9 m thick, and the specified borehole geometry - which included an obligatory tertiary phase to reduce borehole spacing is down to just 1.5 m, with provision for quaternary holes to achieve the final closure. The value selected, and modified after the full scale site trial, was a GIN value of 770 per m.

All injections were carried out with piloted pumps and real time graphical plotting of key injection parameters,

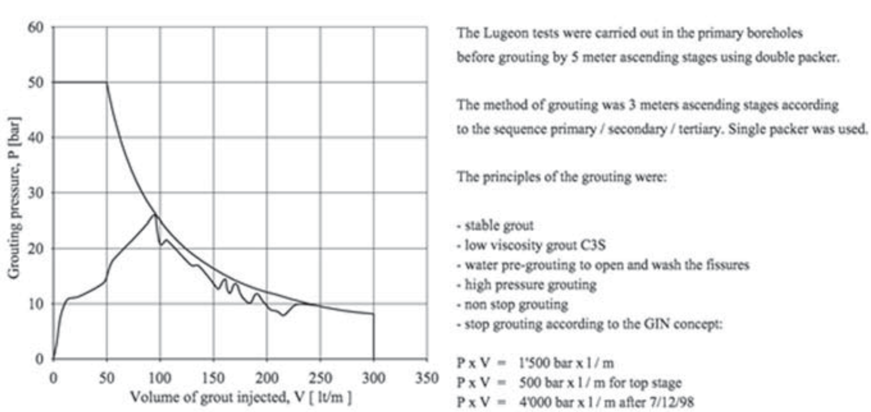


Figure 10. GIN curve & grouting principles for Symvoulos Dam grouting.

including the Equivalent Lugeon value, against both flow and time.

Although a high value had been selected on the basis of the above criteria, the final average grout take was just 311 litre per linear metre for the trial panel (in heavily fractured zone), and 121 litre per linear metre the curtain as a whole. This demonstrates very clearly the self-regulating nature of the GIN process, and its efficiency, limiting flow rates and pressures in response to actual ground conditions. No quaternary boreholes were required.

Following completion of the curtain grouting, an extensive programme of permeability testing was carried out by an independent contractor using sophisticated instrumented double packers equipped with multiple pressure transducers.

The table on the previous page confirms that residual permeabilities recorded within the plane of the curtain were of the order of 10^{-8} m/s to 10^{-9} m/s, with one value of 1.5×10^{-7} m/s. grout dispersal was controlled with no evidence of wash-out towards the river.

Table 1 and the seepage graph shown in Figure 7 indicate the effectiveness of the 2003 and 2014 grouting operations in rapidly reducing the flows through the curtain, and the residual flows reduced to a historic low for the dam, and the distribution of grout plotted graphically, and updated automatically on a daily basis.

Case History 4 - Symvoulos Dam, Cyprus, 1999

Symvoulos dam is located on the British Army complex at their main base on Cyprus. Remedial works at the dam were executed in 1999 to try to bring the reservoir to full serviceability. It had previously proven impossible to impound the reservoir over a period of 20 years due to significant seepage through fissures and discrete solution channels in the soft limestone bedrock.

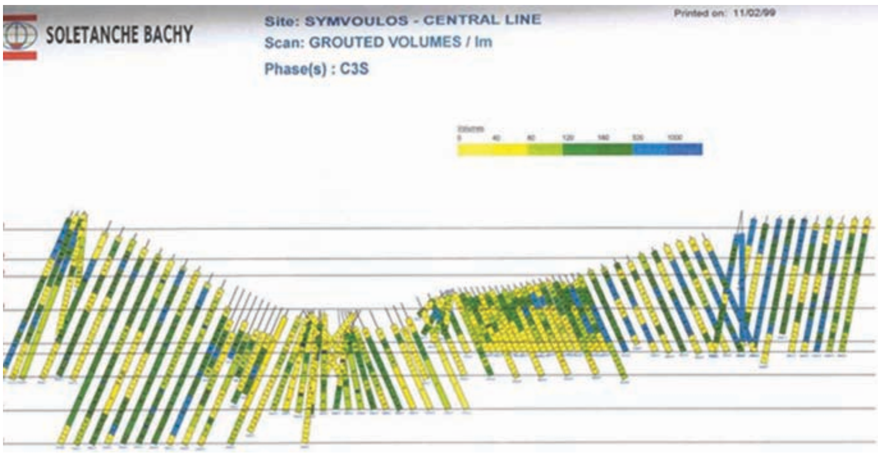


Figure 11. Sketch of grout volumes.

In addition to the original grout curtain, remedial grouting of the foundations using classical stage grouting methods with multiple grout mixes of varying water-cement ratios, had been attempted on three or four occasions of the 20 year period, without success.

In 1999 Soletanche was engaged for a further attempt to seal the reservoir, and to carry out real-time monitoring because of the sensitivity of the upstream concrete face to differential movement. Grout pressure regulation during injection, and uplift control of the dam facade and foundations were therefore key issues during execution of the works.

The solution called for a stable grout capable of penetrating fine fissures, and also filling larger seepage path and voids without bleed. The mix selected, C3S, was a stable cement-bentonite slurry based upon locally sourced ordinary Portland cement, de-flocculated bentonite slurry, and fluidifier.

It was decided to use ascending stage grouting with water flush drilling, and to execute pre-saturation of the rock mass, which had dried out rapidly after the drawdown of the reservoir. It was noted during pre-saturation that pressures of 10 bar or more resulted in dilation of fissures and increased flows. Because of the sensitivity of the structure, and its concrete facade, it was decided to execute the new grout curtain at the upstream toe of the dam, and to restrict pressures with grout to below 10 bars.

Because of the regular geometry of the solution features, and evidence of numerous sub-vertical fissures, a three-row grout curtain was designed with inclined boreholes to ensure that all fissures and solution channels were intersected within the thickness of the grout curtain.

The GIN technique was used throughout for the grouting, but with a stripped down very simplified control system with capacity for piloting 8 pumps.

The works were executed with fully piloted pumps and the results displayed graphically in real-time and updated continuously on the 2-D Model as the works progressed. Final equivalent Lugeon values we used to control the progress of the works, and will reduce progressively from in excess of 30 on the primary holes to below 2 Lugeon on completion of the tertiary holes of the final row.

The outcome of the works was extremely successful and efficient, requiring an average of just 175 litre per linear metre to complete the works (Figure 11). A total of 647 m³ of grout was injected over the course of the injection program.

Grouted volumes were progressively reduced, from in some cases over 1’000 litre per linear metre to below 40 litre per linear metre in the later stages of the works. The higher value absorptions were sometimes isolated, but there was a concentration below the spillway on the right abutment, where numerous voids were encountered during drilling.

On completion of the works the reservoir was successfully impounded and brought into service.

Clif Kettle
Bachy Soletanche Ltd.
Tom Dando Close
Normanton Industrial Estate
Normanton, UK WF6 1TP
E: clif.kettle@bascol.co.uk

Maren Katterbach
Lombardi Consulting Engineers Ltd.,
Via R. Simen 19
C.P. 1535
Minusio, CH 1648
T: +41 91 735 3163
E: maren.katterbach@lombardi.ch

An industry self-evaluation on geotechnical mine closure objectives and planning teams

N. Slingerland, N. Beier, M. Baida

Background

In the spring of 2014, a survey concerning various mine closure goals and typical success rates was distributed internationally to mine closure professionals working in industry, academia, government, and/or some combination thereof. The survey was "high level" in nature, in order to identify trends over time, such as whether certain objectives had evolved or devolved, and also to gauge whether closure goals were being successfully achieved. This type of self-evaluation on a global scale is important from time-to-time in order to assess how we are performing as an industry and where more effort needs to be expended. In a profession that alters the earthly landscape on a scale visible from space, this type of reflection can have a sobering effect.

Nearly 20 years ago, the first known widespread evaluation of closure goal achievement was undertaken in the form of a checklist completed during the inspection of 57 mines in western North America and interviews conducted with site staff (McKenna & Dawson, 1997). The results were variable, but some of the common deficiencies found are as follows:

- Re-established drainage courses such as rivers and channels had not been designed and/or constructed for large enough storm events, nor had naturally occurring blockages such as beaver dams or ice build-up been taken into account.
- End-pit-lakes are still a reality for many mines, and there were several issues such as the reliability of modeling, geochemistry, stratification, and the hydrological performance of these built structures that were identified as requiring further attention.
- Tailings dams that use coarse-silt to fine sand as the primary construction material are highly erodible and rills, gullies, and depositional fans were repeatedly observed. Ongoing maintenance to achieve stabilization of these landscape features is an unsustainable practice, and yet it was also the only approach that was being employed with success.

These deficiencies outline a need for the perpetual maintenance of post-mining landscape features, which is in contrast to the overriding objective of mine closure: that being to return land in a self-sustaining, liability-free state to the Crown. Since this inventory 19 years ago, there has been a steadily increasing wave of environmental activism globally, resulting in undesirable publicity directed towards mining companies based on the land and water quality degradation some have left behind. This often overshadows the positive economic contributions that mining operations make to many regions.

Fear of repeated environmental degradation has fueled protests by the public and NGO's, who's anti-mining arguments have been strong enough to keep mining operations from starting, particularly in regions with a well-versed, longstanding, and articulate community (Kahn, Franceschi, Curi, & Vale, 2001). This kind of attention increases pressure on scientists, engineers, and other closure professionals to achieve their closure targets: not only for the sake of a job well done, but also to ensure they are building a strong track record of success in closure and reclamation works to more easily achieve public license on future projects.

In the authors' 2014 survey, similar issues to those found in 1997 were uncovered. This tells us that while scientists and engineers continue to build on their ability to tackle the dynamic geotechnical problems associated with mine closure, the core issues such as erosion control on tailings dams and establishment of drainage courses remain a challenge.

Survey methodology

The 2014 survey sought feedback from mine closure professionals around the world, and once complete 157 usable surveys were reviewed. Respondents were solicited in a variety of methods including one national land reclamation organization that assisted by sending the online request and web address to their membership, and online mine closure discussion groups on professional networking websites like LinkedIn, for example. Closure goals were grouped into four categories: Technical, ecological, land-use, and socio-economic. Along with general population statistics including years of experience and practice region, professionals were asked whether each of the 29 goals in four categories had been included on their sites, and if so, over what time frame it had been a goal and if they had been ultimately successful in achieving the goal.

From the feedback received, we could identify whether each goal was (1) regularly or rarely set for their sites, (2) evolving or devolving in use over time, and (3) to what degree the goal was successfully achieved in a global context. Answers were recorded in terms of respondents' overall experiences throughout their cumulative work histories and specific sites were not listed.

Survey outcomes

A major focus of the survey was the surface integrity of mined land. As such, topography, above-grade landforms such as tailings facilities or waste rock dumps, and surface stability were emphasized.

Below is a summary of the more thought-provoking survey results:

Goal: Elimination of erosion by wind or water

A strong track record of successful closure and reclamation works is required to more easily achieve public license for future mining projects

Goal frequency: 97%

Success rate: 54.5%.

Erosion control has been a longstanding (20+ years) goal, so one must question why it is only successfully achieved about half of the time.

Depending on geographic location and associated climate, erosion occurs to varying degrees due to the action of wind and/or water over land surfaces. Erosion can be a significant problem where highly erodible sands and silts comprise surface layers of the constructed soil profile. These materials provide good infiltration during minor storm events, but during heavier storm events dramatic rills can form. When not promptly patched these rills can rapidly become gullies that inhibit vegetation growth, remove existing vegetation, and require heavy equipment to repair. If left unrepaired, water quality can be negatively affected to varying degrees depending on the tailings exposed: increased sediment load, development of acid rock drainage (ARD), mobilization of heavy metals and/or failure of mining structures are a few possible results (Nicolao, 2003).

Goal: Prevention of contamination of off-site land or water

Goal frequency: 98%

Success rate: 42%

Mining voids, waste rock piles, and mine tailings can result in the contamination of off-site areas through a number of pathways: excess sediment transport, and the mobilization of ARD, heavy metals, and neutral drainage through water pathways for example, to name a few.

There are various approaches to managing these contamination sources - much of which is mobilized by wind and water. In some cases, generation of contamination can be hindered or treated entirely prior to closure, and this means that ongoing water treatment is not necessary. In other cases, vegetated wind screens can be planted prior to mining to reduce wind-blown transfer. Many of these solutions require site inventories prior to the start of mine operations.

Goal: No ongoing water treatment required in perpetuity

Goal frequency: 84%

Success rate: 32%

Although not directly surveyed, this makes one wonder if the remaining 16% of respondents' who did not include this as a goal had accepted the fact that post-closure water treatment would be required in perpetuity, and that those companies had acknowledged their liability and maintenance costs would be infinitely ongoing. The same can be said for the next goal.

Goal: No tailings ponds/wet covers on site in perpetuity

Goal frequency: 55%

Success rate: 34%

In light of several recent large tailings dam failures, there has been a marked move towards dry stack tailings and other options such as co-mixing that are not associated with catastrophic failures. This trend is expected to continue as public awareness and mine size increase.

Goal: Create a geomorphic/ naturalized (versus uniform/ platform-bank) closure topography

Goal frequency: 94% (78%)

Success rate: 44% (53%)

In terms of topography, survey results showed that there is a marked trend towards the creation of more naturalized topographic landforms, and away from the more traditional construction of uniform slopes and platform-bank topography. This movement began in the early 2000's with work by J.M. Nicolao in Spain, Terrence J. Toy in the USA, and Les Sawatsky in Canada.

Naturalized or geomorphic approaches to man-made topography involve the creation of natural analogues through landforming for closure. The aim of this approach is to reduce erosion and sediment transport, and to reduce the intensity of water transmission to receiving water bodies downstream. Traditional methods such as the platform-bank model and associated adaptations seek to control and direct water movement off of the landform quickly, and to achieve geotechnical stability by inhibiting all mass transfer associated with erosion-induced movement of sediment (Nicolao, 2003). Note that the above statistics are not indicative of the performance ability once constructed.

Goal: Physical stability of waste rock dump(s)

Goal frequency: 90%

Success rate: 70%

Waste rock dumps and depositories are a reality in most mines, but some have the ability to produce chemical reaction, and/or develop preferential settlement that can lead to catastrophic geotechnical failures. These have been noted since the early days of mining, but as our understanding of preferential air and water pathways, water holding capacity, etc. has evolved, so too has the reliability and stability of these landscape features.

The high proportion of success reflects the extensive research and trials that have gone into the field of waste rock physical stability. Associated failures are highly visible and have caused loss of life in the past. While water quality concerns may produce severe environmental consequences, they are not always as visible and thus research on chemical stability may have ranked lower on the list of priorities. The knowledge gap in the area of water quality is being narrowed, but a great deal of work still exists to be done here.

Additional considerations and diversified teams

It has long been said that there would be no mining without geotechnical engineers, but it is also true that there would be no mine closure without geotechnical engineers. Geotechnical goals have an accumulated impact as they directly affect the ability to achieve all other goals (ecological, land-use, socio-economic, and other technical goals). For example, if excessive erosion by wind or water is not eliminated, then the establishment of vegetation on that surface will be difficult if not impossible, land-use such as farmland or recreation will be impeded, aesthetics will be compromised, and off-site land or waterways may have increased sediment loading and/or contamination. It is for this reason that so much emphasis has been placed on geotechnical engineering and related components of closure over the years.

At the same time, it is important to understand the assumptions from which geotechnical engineers base their decisions, and the impact that other components have on geotechnical features. For example, one assumption is that waste rock dumps and tailings impoundments remain constant over time aside from slight consolidation; in this respect ecologists and geochemists will readily argue that soil properties in these landforms alter greatly over time (DeJong, Tibbett, and Fourie, 2014). Input from non-geotechnical professions can inform how one approaches geotechnical problems by fundamentally changing assumptions, so collaboration can be key.

Geotechnical factors have an accumulated impact on mine closure performance

Along the same lines, an interesting finding from our survey was that the success rate of a particular goal is not always directly aligned with professionals for that particular area of specialty; for example, the greatest success rate in achieving ecological goals did not necessarily correspond to the presence of ecologists on closure teams. This tells us that it is the composition of a team overall that leads to higher success rates, not just one profession.

An unexpected finding was that of all professions surveyed, teams with landscape architects were found to have the greatest proportion of successful outcomes: almost double that of any other profession. It is important to note that the broad nature of our survey made it impossible to determine whether this was a cause and effect relationship; however, it is an interesting correlation to say the least. One hypothesis for this is that the generalist nature of the landscape architecture profession ensures oversight such that gaps between specialists' realms are filled. Another hypothesis is that sites with greater closure budgets have the ability not only to do a more thorough job, but also to hire more diversified closure teams.

This is a correlation that has not gone unnoticed: on March 30, 2016 the University of British Columbia hosted the 'Landscapes of Extraction Roundtable' which was sponsored by BGC Engineering Inc. and UBC Sustainability, and organized by Dirk Van Zyl (Norman B. Keevil Institute of Mining Engineering, UBC), Gord McKenna (BGC), Joe Dahmen and Kees Lokman (School of Architecture and Landscape Architecture, UBC), and Mickella Sjoquist (Master of Landscape Architecture Candidate, UBC). The event brought together members of both the mining and landscape architecture community from academia, industry, and with input from indigenous relations specialists. UBC is strategically positioned to develop these discussions, having highly regarded mining engineering and landscape architecture programs, as well as a number of headquarters of international mining companies located in close proximity.

A number of conclusions were drawn from the Roundtable, most notably that:

- The mining life cycle is currently not a cycle at all, but a line. We need to close the gap through better closure work and resultant landscapes.
- The reuse of land is not a specialty of mining companies. Perhaps mining companies should remain focused on mining, and land-use specialists should focus on closure planning aspects, in collaboration.
- If landscape architects are to be involved, the focus needs to be on landscape performance with aesthetics being a natural result.

- Aboriginal communities are being impacted by the cumulative effects of altered land, not solely mining, so a regional land-use approach is necessary in planning
- The term 'Lifescapes' was quoined referring to the constant use of land from pre-mining (this may be difficult to identify with untrained eyes) to post-mining periods, implying a continuum into the future and tying together people and place. It was similarly agreed that the term "closure plan" does not adequately reflect what such a plan needs to do – 'Sustainability', or 'Life Cycle' Plan may be more appropriate.

The Roundtable concluded with a consensus that there is potential for the landscape architecture profession to fill the gaps where traditional mining professionals are lacking in closure planning. The group of approximately 35 representatives that took part closed with the intention to test their collaborative potential on a study site through a design charrette (a graphic brainstorming session frequently used by architects to generate ideas) in the near future.

Conclusions

The landforms that nature has developed over millions of years are in a state of equilibrium; if our objective is to do the same for post-mining sites it makes sense to mimic this example albeit over an expedited timeframe. This idea appears to be catching on: the survey results show a slight trend towards more naturalized approaches as opposed to heavily engineered or controlled methods. One of our challenges as an industry is that mine closure is not typically a revenue generator, and traditional approaches are far less costly than "naturalized" ones. Landform grading is one of the greatest expenses in a closure budget, but it also has one of the greatest (positive) impacts on future performance and maintenance costs. Unfortunately, "form follows function" has been replaced with "form follows profit" as a recurring guiding principle of reclamation works, and this may contribute to the poor achievement: the average success rate was 47.5% for geotechnical goals.

Events such as the 'Landscapes of Extraction Roundtable' epitomize a new trend in closure planning, bringing together mining and land-use professionals to design improved post-mining landscapes

With the exception of avoiding ongoing water treatment and maintenance of tailings impoundments, all goals have been in existence over the last twenty years. Why then, are our achievement rates for these goals so low? One theory is that poor success is not a symptom of inability to achieve the performance target, but a reflection of poor transfer of knowledge, implementation follow-through, and post-closure monitoring. Another theory is that while these individual goals are identified, the sub-goals relative to various specialists (pedologists, hydrologists, geotechnical engineers, etc.) have not been well defined, suggesting that corresponding techniques and approaches may be equally ill-defined.

Regardless of survey interpretation, one conclusion is clear: closure goals are created to protect human and environmental health into the future, and our track record over the last twenty years has not significantly improved. Naturalized solutions appear to be gaining ground, but there is always room for new innovations and approaches when the industry's public license to operate is at stake.

Events like the 'Landscapes of Extraction Roundtable' which draw on the expertise of land-use professions demonstrate a willingness in industry and academia to delve outside of their comfort zones in the search for more effective and meaningful mine closure solutions. While the inclusion of such professionals is not guaranteed to produce better results, this is a radical shift from traditional approaches that are more insulated in nature. A business case for closure work involving naturalized landforms and land-use professionals will likely need to be substantiated in order to drive widespread action.

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N. Slingerland
Department of Civil & Environmental Engineering, University of Alberta. E-mail: slingerl@ualberta.ca

N. Beier
Department of Civil & Environmental Engineering, University of Alberta.

M. Baida
Wax Design, Australia

The 14th Annual University of Alberta Applied Geotechnical Engineering Reinforced Soil Wall Design Contest

Jeffrey Journault and Vivian Giang

The 14th Annual University of Alberta Applied Geotechnical Engineering Reinforced Soil Wall Design Contest was held on April 5, 2016. Sponsored by the Geotechnical Society of Edmonton, the event had five teams of students from the University of Alberta Geotechnical Engineering graduate program and NAIT Civil Engineering Technology program compete for prizes for the strongest wall, closest prediction of wall strength and best design presentation.

Teams of three to five students had one hour to construct a reinforced soil wall using only five sheets of newsprint, two sheets of 15 x 30 cm geotextile, 100 paper clips and 20 popsicle sticks. Construction tools provided on the day of the contest included two rubber mallets, two scoops, two 300 mm pieces of 2 x 4", one 300 mm piece of 4 x 4", one pair of needle nose pliers and one ruler.

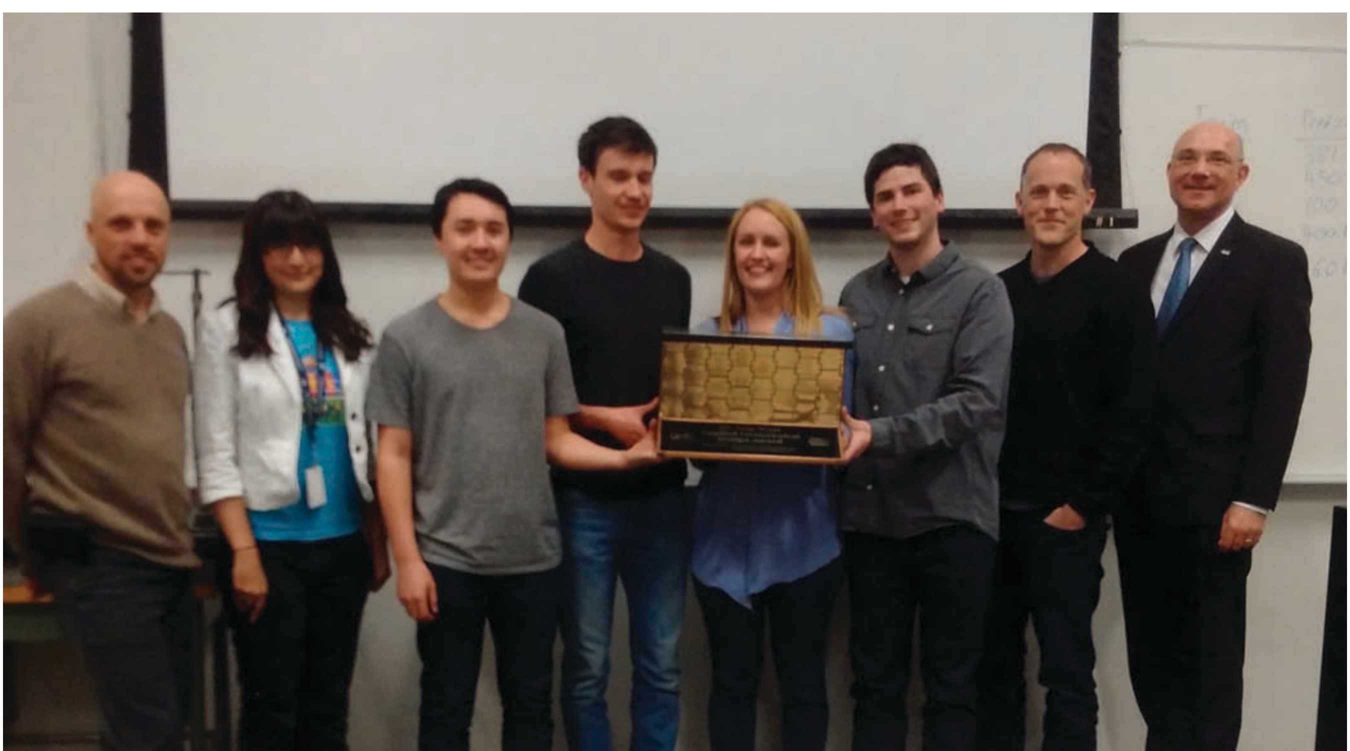
Walls were initially loaded to 200 kPa, with subsequent loads applied in 50 kPa increments. Each load increment had to be sustained for at least 10 seconds, and wall failure was defined as when sand exits from the face of the wall. Winners of the contest included:

Strongest wall (Dr. J. Don Scott Applied Geotechnical Design Award): James Bartz, Hugh Gillen, Haley Schafer and Stephen Lanyi (400 kPa)

Closest prediction of wall strength: Amir Hossein Haghi, Sai Deng, Shuai Gao and Mujtaba Khidri

Best design presentation: Nathalia Ardila, Juan Alejandro Arias, Adriana Luis and Juan Sebastian Gomez

The University of Alberta Geotechnical Centre thanks the Geotechnical Society of Edmonton for their generous support of the annual competition as well as AECOM, BGC Engineering and Thurber Engineering for providing additional contest prizes.



Winners of the Dr. J. Don Scott Applied Geotechnical Design Award with judges from the Geotechnical Society of Edmonton.

History: how it was decided to use a soil-bentonite mixture to seal the pipes passing through compacted clay liners

Robert P. Chapuis

Abstract

Two compacted clay liners were built to retain household wastewater. Just after construction, in the 1980s, full-scale leakage tests, compulsory in Quebec, were carried out. The total leaks were five times higher than target values. The author's analysis suggested either leaks around pipes, or damage to the upper parts of the liners. Field inspection confirmed the two suspicions. The liners were repaired and passed new full-scale leakage tests: they still have a good performance. The solution was to coat the pipes with a 30-cm thick soil-bentonite mixture, to avoid gaps around the pipes due to different thermal dilation factors. This solution was subsequently used for hundreds of clay and soil-bentonite liners after this incident and its successful repair.

Introduction

Compacted clay liners are frequently used for waste storage facilities. For example, in Quebec, Canada, about 1500 cells and lagoons have been built, mostly between 1980 and 2000, to contain solid or liquid wastes. Most liners were made of compacted clays and soil-bentonite mixtures (Chapuis 2002). The total leakage rate of a soil liner depends upon many details in the design, construction and field control of the liner. Several methods have been proposed to predict the leakage rate, using laboratory or in situ test data (Harrop-Williams 1982, 1985; Bogardi et al. 1989, 1990; Fenton et al. 2013), but regrettably without measuring the total leakage rate, a test that is compulsory in Quebec. These methods usually do not consider that poor filter materials and minor construction defects can considerably influence the leakage rate (e.g., Chapuis 1990a, 2002; Guyonnet et al. 2003).

There is a great need for better forecasting the total leakage rate, especially when local bylaws do not require a total leakage test, or when there is not enough water to fill a lagoon. The test was enforced in Quebec in the early 1980s (e.g., SQAE 1985) in addition to other field tests (Chapuis 1990a, 1995). It is the last field control required before final approval of the liner: it is used to compare prediction and reality. In Quebec, over 1000 lagoons were tested for total leakage (liners of surface between 10^4 and $5x10^5$ m^2).

Here, the story of two clay liners, built in the 1980s and immediately tested, is presented. The total leakage was five times higher than the target value. A simple analysis in the 1980s yielded a correct diagnosis and led to efficient repairs. The lagoons passed the leakage tests after repairs, and have had a low leakage rate up to now. The total leak, when a lagoon is in operation, can be measured in the hydraulic control structure of the drainage system (filter sand, drain pipe, solid pipe) below the liners.

The predicted outcome was correct, but unpublished. Lessons learned have not been passed on. New analysis and predictive methods for compacted clay liners were proposed later (Chapuis 1990a; Chapuis et al. 2006; Chapuis 2013), long after this case of poor performance.

In this paper, basic rules for design and construction are summarized and then, the full-scale leakage tests of the 1980s are presented, including the analysis which yielded correct diagnosis and successful repairs. The reader may be curious as to why such old data are only now being published. The case resulted in a dispute, which ended with a confidential out-of-court settlement, implying that nothing could be published. This was regrettable because valuable technical information was retained or hidden. Over the past few years, the author has requested authorization to publish scientific analyses of old, but still interesting, cases. As time heals all wounds, the authorizations to publish arrived, with the condition that all names, dates, and legal issues be kept confidential, and that no photograph should permit identification of a site or person.

Design and construction of compacted clay liners

For lagoons in Quebec, soil-bentonite liners are 10–25 cm thick, whereas clay liners are 45–90 cm thick. Soil-bentonite liners are constructed in a single lift, whereas clay liners are constructed in several lifts to avoid superposing defects and joints. The total leakage rate must be lower than some target value, and the liner must have adequate mechanical properties (e.g., Goldman et al. 1990; Benson et al. 1999; Camp et al. 2009).

Laboratory tests have shown that the hydraulic conductivity, *K*, of compacted clay is influenced by the moulding water content, w_c, and degree of saturation, S_{rc}, after compaction. Specimens compacted wet of optimum are usually more impervious than those compacted dry of optimum with the same energy (Lambe 1954; Mitchell et al. 1965). Two types of porosity coexist in compacted clay (e.g., Li and Zhang 2009; Romero 2013; Della Vecchia et al. 2015). The primary porosity corresponds to the fine, micron-scale, structure of solid particles within the clods. The secondary porosity is due to poorly interlocked clods and lifts after compaction. The *K* value of compacted clay, once saturated, depends mostly upon secondary porosity, as shown by dye-stained seepage (Elsbury et al. 1990). Compaction wet of optimum corresponds roughly to S_{rc} values close to 90%, which means that 10% of the void space is filled with trapped air which is difficult to remove by compaction. The resulting low air permeability was used as a field control of compaction conditions (Langfelder et al. 1968).

For clay compaction the dry density, ρ_d, must be equal to or higher than a given percentage of the optimum value, ρ_{opt}, of the standard or modified Proctor test. The two optima are well correlated to each other (Chapuis 2002). In the 1980s, the minimum moulding water content, $w_{c,min,}$ was the optimum water content, w_{opt}, whereas after 1990 for Quebec it became the plastic limit, w_P. Other specifications (after 1990) may include a minimum value of S_{rc} of 90% and also some maximum, w_c, value, $w_{c,\, max,}$ to permit normal equipment traffic. Other criteria are the liner total thickness, the thickness of each lift (15–30 cm), the length of the steel feet on a sheep foot roller used to knead a clay lift, and sometimes, a maximum clod size (Benson and Daniel 1990).

In Quebec, most liners before 2000 have been built with local clays, which are not fissured in natural deposits, except in their upper crust. These natural clays lose their sensitivity, if any, after drying, which allows their use in liners. All clays are mostly rock flour (Foscal-Mella 1976; Locat et al. 1984). The *K* value of natural (in-situ) non-fissured clays is usually in the 10^{-10}–10^{-9} m/s range (e.g., Tavenas et al. 1983; Duhaime et al. 2013; Duhaime and Chapuis 2014), thus lower than the *K* of compacted clays in the 10^{-10}–10^{-7} m/s range (Chapuis 1999; Chiasson 2005).

Sufficient compaction of each clay layer is verified by compaction control tests, using neutron probes or other methods. In addition, to retain its quality, the liner must not dry or freeze before being used. This is a major issue in Canada. Otherwise, the *K* value could increase by two orders of magnitude, and other properties may be affected (Chamberlain and Gow 1979; Kim and Daniel 1992; Benson et al. 1995; Eigenbrod 1996; Chapuis 2013; Xue et al. 2014). As soon as the clay liner is constructed, in Quebec, it is covered with 20–30 cm of sand or sand-and-gravel that is kept moist to avoid detrimental effects of desiccation (Albrecht and Benson 2001; Yesiller et al. 2000). The use of geotextile may also help to decrease cracking (Safari et al. 2014).

Other field controls include permeability tests in specially installed monitoring wells, or long-term infiltration tests (Day and Daniel 1985; Chapuis 1990a, 1999; Guyonnet et al. 2003), which are time-consuming and can be carried out only at a few places. When the liner construction season is short, as in Canada, only a few tests can be performed. In addition, experience in Quebec indicates (unpublished results) that the few *K* values, “measured” at a few places, poorly predict the actually measured and monitored total leakage rate.

Many case histories, from several countries, were published with little information other than reporting a too high leakage rate. However, technical reports by engineers and technicians fully document compaction conditions and *K* values as determined by laboratory or field tests. The liner performance may then be predicted from statistics based on these tests, but alas the total leakage is usually not measured. This seriously limits our ability to confront prediction and theory. In a few published cases, the total leakage rate was simply said to be 10 to 1000 times higher than predicted (Auvinet and Hiriart 1980; Daniel 1984; Picornell and Guerra 1992; Chapuis 2002). A few authors have tried to explain this difference by large-scale effects. Their opinion is that full-scale tests are more likely to contain preferential flow paths, and thus yield large-scale *K* values higher than the *K* of smaller-scale tests (Shackelford and Javed 1991; Cazaux and Didier 2002). However, the statistics for large sets of compaction control data reveal lognormal distributions: these may then be used to predict the full-scale *K*, for the total leakage rate, which eradicates the need to invoke scale effects (Chapuis 2013).

The project as built and repaired in the 1980s

The two rectangular lagoons were built, tested and successfully repaired in the 1980s. Their bases had areas of 130 m x 50 m and 130 m x 40 m respectively. Their sides had a 1V/3H slope. Each liner was 75 cm thick, built in five 15-cm thick lifts. The liners were constructed in July, during a warm, dry, summer. The completed clay liner was covered with 20 cm of gravelly sand on the bottom and 30 cm of crushed stone (0- 20 mm) on the slopes. The equipment for wastewater treatment was installed. Four pipes, which carried influents and effluents, passed through each liner. A photograph of a representative lagoon, half-full of wastewater, is shown in Figure 1.

The clay had a mean plastic limit w_P of 26%, a natural water content,

Figure 1. Example of half-full lagoon for wastewater treatment (photo by author). The pipe (bottom right and along the crest of the dike) provides air to the aerators, the upper part of which emerges in the partially filled lagoon. Two influent or effluent pipes can be seen above the water line.

w between 36% and 41%) below the liquid limit, w_L, and a specific gravity of solids, G_s, of 2.78. A standard Proctor test gave an optimum, ρ_{opt} = 1587 kg/m^3 at w_{opt} = 23.8%. The specifications required that $\rho_{dc} \geq 90\% \rho_{opt}$, and $w_c \geq 23.8\%$, after compaction. There was no specification for S_r after compaction, S_{rc}. The total leakage rates had to be lower than target values defined as a water level drop of 1 cm/d in lagoon 1, and 2 cm/d in lagoon 2. The different values were related to the environmental impact evaluation made at the time (1980s) by the designing engineer.

The total leakage rate of each lagoon was measured by monitoring the water level versus time after the valves on influent and effluent pipes were closed. The water levels were measured to the nearest mm within hydraulic structures connected to the lagoons. This eliminated wave effects. The levels were corrected for rain and evaporation, using rain gauges and evaporation pans.

None of the liners passed the test. For lagoon No.1 at full water level, the drop was 5 cm/d, five times higher than required. For lagoon No.2 at full water level, the drop was 10 cm/d, five times higher than required. Leakage rates were measured at different water levels in the ponds.

Full-scale leakage test – lagoon no. 1

The water level versus time is shown in Fig. 2a. The total flow rate is noted Q for a water height h above the top of the bottom liner. It takes a maximum value Q_{max} for a maximum value of h, h_{max}. The ratio Q/Q_{max} versus h/h_{max} is plotted in Fig. 2b. The theoretical curves in Fig. 2b are those given by the closed-form equations of Chapuis (1990a), which involve two hydraulic conductivities, K_b for the bottom part and K_s for the sloping part of the liner. The initial water level drop was about 5 cm/d, 5 times higher than the target value. Figure 2b helps to identify the nature and location of hydraulic defects. The field results (Fig. 2) could have three explanations:

1. The liner could have been fissured when the water level exceeded a certain elevation. This can occur if the liner rests on a low bearing capacity soil: the large uneven settlement creates fissures in the liner. The cracks stay opened when h/h_{max} exceeds a certain value, but may close when the water level drops. For the two lagoons, however, the underlying soils were dense till with a very small settlement, which would lead to discard this first explanation.
2. According to Fig. 2b, the upper portion of the sloping liner was too pervious. In this case, the leakage rate depends on the difference in elevation between the pond surface and the bottom of the damaged zone, which would explain the shape of Fig. 2b. This explanation (damaged upper portion) was proposed in the 1980s, after it was noted that the half-full lagoon leakage was much smaller than half of the full lagoon leakage.
3. The elevation at which the leakage seemed to vanish was equal, within a few centimetres, to that of the base of the influent and effluent pipes crossing the liner. Therefore, the shape of Fig. 2b could also be due to preferential leakage along poorly sealed pipes. This explanation (poorly sealed pipes) was proposed in the 1980s after noting that the leakage rate nearly vanished when the lagoon was half-full, thus for water below the elevation of the pipes.

Full-scale leakage test – lagoon no. 2

The water level versus time appears in Fig. 3a and Q/Q_{max} versus h/h_{max} in Fig. 3b. The initial water level drop was about 10 cm/d, 5 times higher than the target value. Fig. 3b shows that the leakage rate seems to vanish when h/h_{max} reaches 37.5%. The discrepancy between predicted and measured leakage rates could be due to the same three reasons as for lagoon No. 1. Here again, the elevation at which the leakage seemed to vanish was equal to that of the base of the pipes crossing the liner. The first reason (settlement) was discarded but the

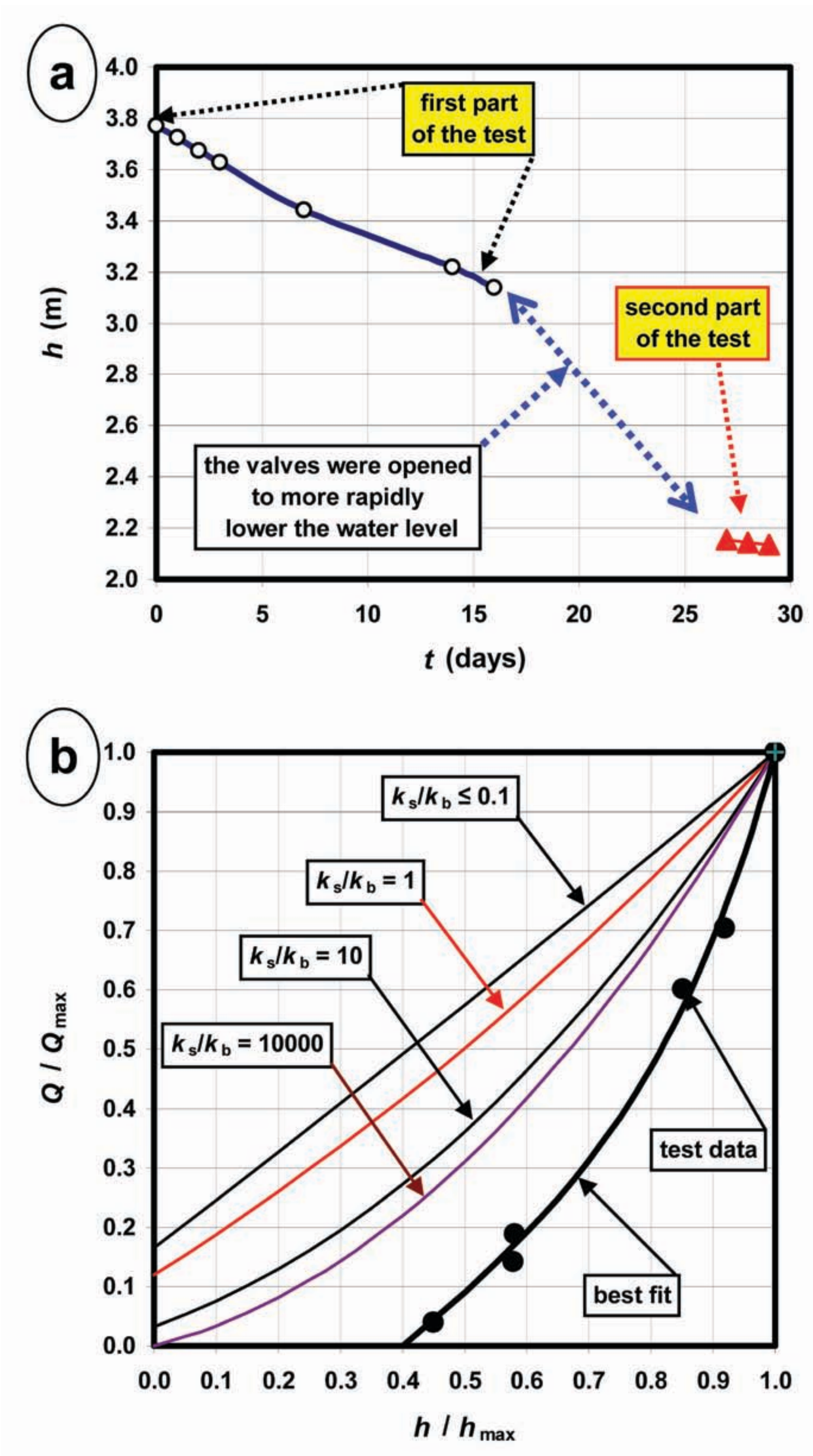


Figure 2 Full-scale leakage test, lagoon 1: (a) water level versus time; (b) non–dimensional graph of relative flow rate Q/Qmax versus relative water thickness h/hmax for the same lagoon geometry.

two other reasons were retained in the 1980s field investigations following the poor performance of the liners.

The results of the two full-scale leakage tests showed that the leakage was reduced when the water levels reached mid-slope. Because the water thickness above the liner had been halved, the leakage rate should have been roughly halved. However, this was not the case. Leakage rates were only 1-2 mm/d (these small values were inaccurate) and thus, much less than half the leakage rate at full water level. These findings led the author, acting as an expert in the 1980s, to suspect that mid-slope pipes were improperly sealed or the upper parts of the liners were too permeable.

A possible indication of damage in the upper parts of the liners was found in the July report of the construction inspector who reported cracking in the upper slopes. The engineer then requested a verification that the cracks would not be more than 3 to 5 cm deep, otherwise the contractor would have to spray water to increase the clay water content, re-mix the 15-cm clay lift, and re-compact it. This may have been done, but there was no written evidence of this in subsequent field reports. In addition, the two lagoons did not receive rain water or water spraying during weeks between the end of their construction and the full-scale leakage tests. This situation was physically detrimental to the liners.

A few weeks after the full-scale tests, all participants in the project agreed to empty the two lagoons. This was needed so that the reasons for the poor performance could be investigated.

Field verifications of the liners and pipes

After the full-scale leakage tests, the lagoons were emptied for inspection. Shelby clay samples were taken in the upper slopes. The liner thickness in the upper slopes, supposed to be 75 cm, was only 45 cm on average for liner No. 1, and 40 cm for liner No. 2. The measured values ranged from 29

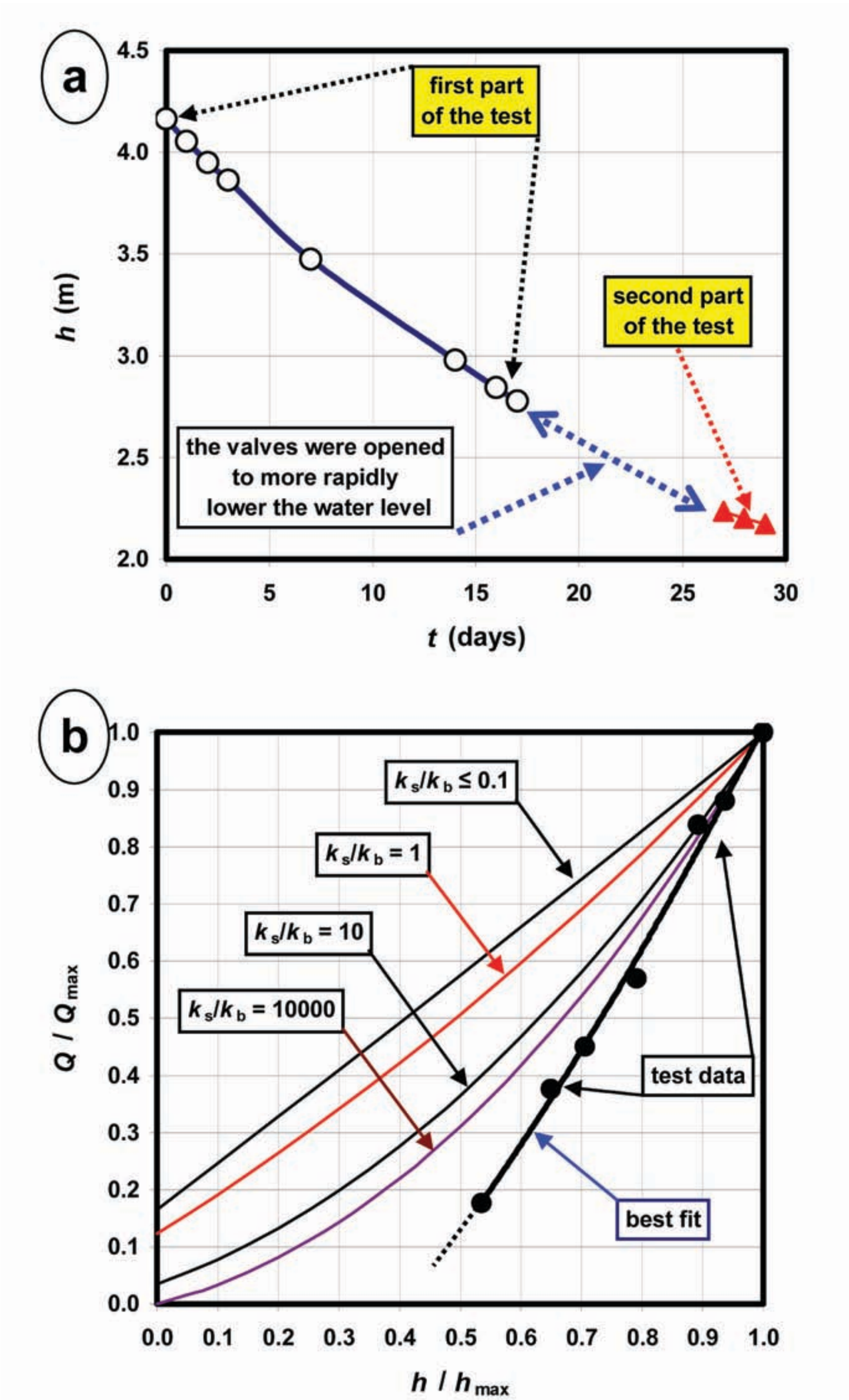


Figure 3. Full-scale leakage test, lagoon 2: (a) water level versus time; (b) non-dimensional graph of relative flow rate Q/Qmax versus relative water thickness h/hmax for the same lagoon geometry.

to 55 cm for liner No.1, and from 23 to 59 cm for liner No. 2. The samples had a few subvertical cracks crossing the full thickness. In addition, 30% of clay samples contained thin, less than 1-mm thick, layers of sand within the clay liner, probably resulting from poor handling of materials during construction. Therefore, the first suspected reason for poor performance, some clay damage or poor quality work in the upper parts of the slopes, was verified.

The second suspected reason, a poor seal along the pipes, was also investigated. During a site visit, when the water levels had been lowered, the author could see open spaces just below the pipes with widths between 5 and 8 cm. The spaces were sounded using a rod: they were opened along the full liner thickness. Smaller (1-2 cm) open spaces were also found above and around the pipes, after the crushed stone had been removed to expose the contact between clay and pipe. A representative pipe is shown in Fig. 4. Thus, the second suspected reason was also verified.

Reason for the poor seal along the pipes

Initially, the clay had been tightly compacted against the pipes, which had been installed during dike construction and before liner construction. Subsequently, a gravel protection was laid everywhere over the liner, with a special shape and increased thickness around the ends of the pipes, as a protection against erosion. The open spaces that were found along the pipes may have been caused first by thermal dilation and contraction, and then enlarged by water erosion.

The pipes were laid in June. The field inspector reported that the contact between clay and pipe was good. During its installation, a pipe was at a temperature close to that of the newly placed clay (≈ 10°C). However, in the next few weeks, its temperature increased significantly (from about 10 up to 35°C), because the empty pipe was in contact with the hot air inside it. All pipes were made of plastic. Their thermal dilation coefficient is about 12 times higher than that of cement concrete or steel (Chapuis 1990a). Therefore, the plastic pipes dilated, their wall pushed away the clay, thus tightening the seal at the

Figure 4. Photograph of the poor contact between a pipe and the clay liner (photo by author). A large opening appears as a black crescent below the pipe.

interface. During the autumn, the lagoons were filled with cold water and the temperature of the pipes decreased to 5-10ºC: the 25-30ºC change made the pipe diameter decreased by 1-2 mm for pipes having a diameter in the 30-60 cm range. Unfortunately, the clay used for the liner had no swelling capacity: it was unable to follow the pipe thermal contraction. As a result, there was a small annular space around each pipe, and some arching effect in the clay. The pore space existed when the lagoons were filled (cold water in autumn) and during the full-scale leakage tests. Thus, a preferential leakage started in the annular space. This leakage eroded the clay and enlarged the initially small space, up to 5–8 cm, during the time the water level fell from its maximum elevation down to the pipe elevation, where the leak around the pipes went dry.

This clay liner project provided a lesson for subsequent projects in Quebec. The design of seals around pipes was modified. The next seals were made using a rich soil-bentonite mixture, 30- to 50-cm thick. When the dry mass of bentonite is 16-20% of the total dry mass, the mixture can follow the pipe thermal contraction and dilation without losing the hydraulic seal. Such a solution appears in a photograph for a much larger project with large pipes (see Fig. 8 in Chapuis 2002). In his files, the author has over twenty, mostly unpublished, cases of failure for soil-bentonite liners and compacted clay liners, but this case (one of earliest cases of liner construction and liner failure in the author's files) was the first and last one with a sealing defect around plastic pipes.

Reconstruction in the mid-1980s

The upper parts of the liners were rebuilt with the same clay. The field geotechnical control was continuous instead of part-time during initial construction. For reconstruction, it was suggested that the pipes be sealed with a soil-bentonite mixture. This suggestion was followed by the consulting engineer and used by the contractor.

Immediately after the repairs, the two lagoons were filled, up to their top levels. All valves on the pipes were closed, and the leakage rate of each lagoon was measured. The two liners passed the new tests and were accepted. Since the 1980s, they have performed well, as indicated by their low leakage rates, which can be monitored in the filtering-drainage system below the liners.

Availability of new predictive and control tools

In the 1980s, predictive and control tools had serious limitations. This was an incentive to develop closed-form solutions for analyzing full-scale leakage tests and detecting the position and stability of different types of hydraulic defects. Closed-form solutions were developed and then verified with a few poorly performing liners. They provided correct diagnoses, and all liners were successfully repaired. Afterward, closed-form solutions were published (Chapuis 1990a, b). These were also used to predict the infilling rate of shallow lagoons which are filled slowly once a year, then emptied, and for which meteorological conditions, which can bring 50 cm of rain water or snow water, may be critical (Chapuis 1991a, b). Later, equations were developed to predict the *K* value of compacted clay (Chapuis 2002, 2012; Chapuis et al. 2006): these equations use field compaction data to predict a *K* value at each place a compaction control was carried out. Recently, a statistical method was also developed (Chapuis 2013) to predict the full-scale leakage of a liner, using the small-scale *K* values predicted with compaction control data.

The 1990 closed-form solutions were proven correct in a few published cases (Chapuis et al. 1992; Chapuis 2002), but the author has only published a few of the many failure cases for which he was an expert. The closed-form solutions for the local values of *K*, and the resulting large-scale value of *K*, were verified recently (Chapuis 2013) for a case of frost damaged liners. More case studies are needed for full proof of correctness. The compaction data of the two clay liners of this paper, built and successfully repaired in the 1980s, provide an opportunity to test the new predictive methods, because they permit direct comparison between predicted and measured total leakages. This detailed comparison will be presented in another paper.

Discussion and conclusion

Two compacted clay liners were constructed, tested for full leakage, poorly performed, but were successfully repaired in the 1980s. Both liners failed the first full scale leakage tests. Each liner had a total leakage rate about five times higher than the target value. The leakage, measured at several water levels in the lagoons, almost vanished at mid-water height. This led to suspect either a poor condition of the liner upper parts, or some leakage around plastic pipes installed at mid-height of the slope. Further investigation has confirmed construction defects in the slopes and found major holes around the pipes. The defects were repaired soon after; the liners were tested again and passed the new total leakage tests. This confirmed the diagnostic correctness and repair success. Thirty years later, the clay liners are still performing well, according to their low leakage rates as measured in the drainage systems below the liners.

After these two liners were built and repaired in the 1980s, several methods were developed to more rigorously analyze a full-scale leakage test (Chapuis 1990a), to predict the *K* of compacted clay at each place a compaction control has been done (Chapuis 2002), and to statistically predict the resulting large-scale *K* value of the liner (Chapuis 2013).

However, even if predictive methods are available now, there is still a need to perform full-scale leakage tests, wherever possible, because a few field permeability tests in a clay liner cannot correctly predict the total leakage, and because no predictive method can take into account errors in design and poor field work. In addition, full-scale testing is the only way to compare prediction and theory, which is crucial. Since the 1980s, there has been an undeniable learning process in the design, construction, and field control of compacted clay liners, but there are still lessons and information to be extracted from old case stories. It is also important to publish technical aspects of poor performance cases, because these may help all those involved in design, construction and control of compacted clay liners.

Acknowledgments

The author thanks the site owner who authorized publication of this case history, with the request that all names and legal issues be kept confidential. He thanks Arthur Yelon for his editing help.

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Robert P. Chapuis
Department CGM, École Polytechnique, P.O. Box 6079, Stn. CV, Montreal, Quebec, Canada, H3C 3A7
(514) 340 4711 ext. 4427
Fax: (514) 340 4477
Email: robert.chapuis@polymtl.ca

GEO-INTEREST

Case History IX

Part 1

Hugh Nasmith has put together an excellent book on litigation which is easy to read, covers the litigation scene thoroughly, has subtle humour, and most important of all, is umderstandable. He remarks in the opening paragraphs that experienced geotechnical engineers will find nothing new in the book except comfort that their situation is not unique. This is true but experienced engineers should read it anyway. (From a review by William A. Trow).

This case history is copied almost word for word from the written judgement of the trial judge who heard the case. Where the original judgement gives names of those involved the appropriate terms Contractor, Owner, Engineer, Technician, etc. have been substituted. Although longer than some of the other cases it is valuable because it is clearly written and permits the reader to follow the reasoning by which the judge arrived at his decision.

The defendant is a one-engineer soils engineering firm against which the plaintiff seeks to recover for the failure of a concrete floor in a warehouse on its land which settled because of inadequacies in the design and application of a "preload" of piled sand which had been used to compress the peaty soil in preparation for construction.

The soils engineering firm (which I shall refer to as the "the defendant") was not engaged to design or supervise the preload. Nor was it given the information which it would require in order to express an opinion on the appropriateness of the preloading which the plaintiff did. But the plaintiff says the defendant, though not retained or paid to advise on the matter ought to have known that it was being relied on for advice and had both a duty to take care not to mislead the plaintiff and its contractors and a duty also to warn of danger which it should have foreseen in what the plaintiff was doing.

The present action as originally framed was also against the contractor who constructed the plaintiffs building, but this claim was settled before trial. By agreement between the present parties the court was asked to determine the percentage share of fault, if any, properly attributable to the defendant - as opposed to that attributable to the plaintiff and the contractor for the settlement and failure of the floor, and to assess the amount of damages, if any, properly chargeable to the defendant on the basis of any such apportionment of fault. The terms on which the claim against the contractor had been settled were not disclosed

The Background

The plaintiff, Blank Developments Ltd., is a company belonging to Mr. John Doe and a partner whose affairs, at least in the context of the project in question, have been managed by Mr. Doe.

The building site was acquired for the company in 1975. The intention was to build a warehouse or workshop building there, and to rent out space in the building to a tenant or tenants engaged in light industrial or commercial businesses. Neither Mr. Doe nor his partner had any significant previous experience in construction. They made enquiries about a cement-and-wood building, to be constructed on a wholly-piled foundation, but found the cost-in the neighbourhood of $300,000— too high for the project to pay its way. During 1978 they heard of building alternatives which might make the project economically feasible. They also found out something of the dangers inherent in the use of less costly methods of foundation design.

Early that year Mr. Doe learned that there had been settlement in the floor of a building on the next-door property, and also something of its cause.

This building had been constructed with a piled perimeter foundation supporting the walls and a cement floor "floated" inside on unpiled ground. This foundation design had been adopted against the recommendations of a soils engineering firm retained by the owner. Mr. Doe was shown the soils engineering report in question. Its most significant passage reads:

We understand that you intend to pile support the structure and were intending to "float" the floor. Based on the depth of peat encountered we do not recommend that the floor be supported by any means other than pile support. Site conditions such as these warrant total pile support for the building.

This reference to a "mixed" foundation is significant in the present context. It was this very technique which the plaintiff was ultimately to adopt for its own building. It was to do so with knowledge of the consequences which had flowed from the use of that design in the case of the building next door.

Some knowledge of the way in which "preloading" works is essential to an understanding of the problems which lay ahead for the plaintiff.

In such peaty soil conditions, preloading is generally a less expensive but more time-consuming method of foundation preparation than piling. Done carefully it will eliminate, or at least minimize, the risk of settlement taking place after a building has been erected on the prepared site. The compromise adopted for the neighouring building, and for which the plaintiff was to opt in the end, involves a pile-supported concrete perimeter foundation for the walls with a "floated" slab poured on preloaded soil inside. Engineering opinion is divided as to the wisdom of adopting this mixed foundation design. The evidence suggests that a relatively small settlement, which might be tolerable were the whole building on a "floating" slab, can play havoc if the walls are stabilized on piles and the floor alone is floated on unpiled preloaded soil.

The technique of preloading, while neither particularly complicated nor exclusively within the province of the soils engineer, calls for certain expert attention both in the planning stage and in application.

The amount of sand required for preloading a peaty soil must exceed by an appropriate margin the greatest weight which will subsequently be imposed on the ground which it is to compress. The preload is usually a sand pile shaped, very roughly, in this manner:

The crown of the pile has to extend beyond the boundaries of the actual building site, or "envelope". The sand must be uniformly shaped, so that the site will be uniformly compressed. The load must be kept in place until all settlement has ceased. In calculating the amount of preload applied the engineer must exclude any part of the material which is to be left on site to restore the original ground level after compression, or to raise it to a new elevation. That constitutes part of the weight which the soil must be prepared to carry, not part of the preload. The preload is that portion only of the added material which will be taken off the building envelope after settlement has ceased.

Thus the design of an appropriate preload requires calculation of the weight of the proposed building and contents, the weight of the material to be left in place as fill, and the weight of the material to be removed. The preload must be properly shaped and so placed that this crown overlaps the building envelope. Settlement must thereafter be completed — stability must be achieved before the preload portion of the material can safely be removed.

These are some, at least, of the matters to which the mind of an engineer must be directed in designing and supervising a preload.

When Mr. Doe was looking for an economic solution to his construction problem early in 1979 he must have known that a partially-piled foundation with a floated slab floor on preloaded grade would probably be cheaper than an all-piled foundation.

He knew that such a design —in which the floor area only is floated —had been rejected for these soils conditions by one soils engineer, that it had been proceeded with notwithstanding that advice and that it had failed. While he knew very little about preload, Mr. Doe knew enough to recognize that he would need expert guidance in order to minimize the risks involved if that design should be adopted for his own project.

In May and June of 1979 Mr. Doe discussed his requirements with personnel from the Contractor who provides and erects pre-fabricated steel buildings, and was quoted more attractive prices.

While these discussions with the Contractor originally centred around an all-piled foundation design, the Contractor also mentioned to him the possibility of a "floated" floor. Mr. Doe brought a quantity of sand onto the property and dumped it in individual truck-load piles within the building envelope. He says he did this not for the purpose of preloading, but with a view to raising the level of the site on which he intended to build.

During the discussions between Mr. Doe and the Contractor the representative of the Contractor said they would need to have a soils test done, and recommended that the defendant be asked to do it.

The Preliminary Report

The defendant is a company through which Mr. Smith carried on his practice as a soils engineer with the assistance of three employees — two technicians and a secretary.

It was one of the technicians, Mr. Jones, who answered a telephone call from Mr. Doe on June 12. Mr. Doe described the sort of building he had in mind and said he was planning to build on an all-piled foundation. Mr. Jones said that was a good idea in view of the soils conditions in the area. Mr. Doe said that he was thinking of having the Contractor erect the building and that they needed a soil investigation. He mentioned that the owner of a nearby building had experienced settlement problems. Mr. Jones suggested a three-hole test program as appropriate and said he would get a driller to quote a price and let Mr. Doe know the total cost. After getting the drilling quotation, he phoned back and said the cost would be $900. Mr. Doe phoned later and said they didn't want to pay that much; he asked for something less elaborate. After discussion with Mr. Smith, Mr. Jones quoted $400 for a report on a single test hole, and Mr. Doe accepted.

Mr. Jones went to the site three days later and supervised the test. The nature of the test and the conclusions which the defendant drew from it are described in a document dated June 18, which plays a central role in the present litigation. Headed *Report of Preliminary Subsurface Soil Investigation and Recommendations*, it reads as follows:

Introduction

In accordance with your request a preliminary subsurface soil investigation was conducted June 15 at the above project site. The proposed 50 foot by 100 foot building will be steel frame with metal siding. A pile foundation is planned. This report presents recommendations for the pile support of the foundation and for the slab-on-grade floor.

Investigation

One penetration test hole was placed at the location shown on the attached Test Location Plan. A modified top drive Mayhew drill rig was used to a depth of 50 feet. A 4 1/2 inch diameter auger hole was bored to 12 foot depth to explore the upper soil strata. This hole was placed about 3 feet south of the penetration hole.

Description of Site

The site is uniformly flat. No trees exist. Stockpiles of river sand have been deposited on the building site to a depth of about 9 feet for the purpose of preloading the slab area.

Description of Subsoil

The upper 20 feet of the site is composed of a brown non-fibrous peat with some fibrous peat mixed in. This material is soft and saturated below about 6 foot depth. The peat is mixed with clay from about 20 to 30 feet and probably changes at about 30 feet to a sand and silt which exists to the maximum 50 foot depth explored. This sand provides suitable bearing for piles.

Conclusions and Recommendations

The upper 30 feet of soil is unstable and will consolidate under anticipated floor loads. Preloading is advised to stabilize this soil. The river sand currently on site is suitable preloading material. Use 1 foot of this sand as surcharge for each 95 psf of dead and live load anticipated on this floor. The penetration test indicates that individual size 13 piles (minimum) driven to 50 foot depth will develop 10 tons Allowable Bearing Capacity. The same size piles driven to 60 foot depth may develop an Allowable Bearing Capacity of 20 tons if the sand density increases however this investigation terminated at 50 feet and this increase in density was not substantiated. The piles may be either: used, marine piles, 10 pcf creosoted foundation piles; or green pile for the lower section and creosoted 15foot top section. A securely fastened pipe splice is recommended to join the upper and lower sections of the 2 piece pile. The pile driving operation should be supervised by someone competent in this type of work in order to ensure adequate bearing for the piles on this project.

If questions should arise, please contact the undersigned or Mr. Jones.

Mr. Smith, P.Eng.

The position of the plaintiff is that this report gives the appearance of approving use of the sand there —as dumped in truck-load piles on the site — for a preload, and misled the plaintiff into following that course. The plaintiff says the references to the investigation as "preliminary" contained in the healing and opening sentence of the report, are not sufficient to constitute a warning that recommendations are not to be used for construction purposes.

The report was put into final form and approved by Mr. Smith. It was picked up soon afterwards by Mr. Doe. Mr. Doe read the report, but he says he regarded it as something intended for the contractor rather than himself. So he took it to the contractor's office.

The contractor is a company which has professional engineers on its staff. It supplies and erects prefabricated buildings, with ancillary engineering services, including foundation design and site inspection. It is quite apparent that both Mr. Jones and Mr. Doe intended the soils report to be used by the contractor's engineering personnel, for whom it had been ordered. There is no suggestion that anyone thought it was intended for the guidance of laymen, such as Mr. Doe and his partner.

I have concluded that this report was intended to be "preliminary" in the sense that its purpose was to assist a construction engineer in costing, and deciding between, foundation alternatives. It was not intended to be used for actual foundation construction, though the information concerning the piled foundation was probably adequate for that purpose.

The Design Phase

Sometime during the latter half of June the plaintiff retained the Contractor to supply and construct the prefabricated building and to perform engineering services required for the project.

The Contractor was not to be a "general contractor", in the sense of having total responsibility for the whole work, and actual preparation of the site and foundation construction were specifically excluded from its contract. But the matter for which it undertook responsibility included, among others: "Foundation design including letter of supervision and site inspection" and foundation design drawings, signed and sealed by a registered Professional Engineer. The plaintiff is said to have been "its own general contractor" in the sense that the plaintiff was to arrange, at its own cost, for all work required other than that undertaken by the contractor, including site preparation work and construction of the foundations. But as part of its lump-sum contract the contractor undertook to design the foundations and to inspect the site prior to construction. The contractor was to provide a supervising engineer for the project, in addition to providing and erecting the "pre-engineered" steel building.

The Contractor proceeded with the preparation of drawings. These contemplated in place of the all-piled foundation which had originally been planned, the less-expensive mixed design — a concrete foundation supported by piles for the walls and concrete slab floor poured on preloaded, unpiled ground inside. Before completing these drawings, the contractor's chief engineer telephoned Mr. Jones, the defendant's technician, to ask about preloading

The contractor's engineer, Mr. Brown, asked Mr. Jones how longthe preload should be left in place. He says Mr. Jones replied that it should remain in place for eight weeks or until settlement ceased. Mr. Jones says he replied that he did not know how long settlement would take, that he had heard reports of eight weeks being a sufficient time for settlement to take place, but that the way to find out was to use settlement gauges. Mr. Brown told Mr. Jones he was going to make some reference to preloading in the drawings, but he did not indicate what it was he intended to put on the plan.

I found Mr. Jones a credible witness and his recollection of this conversation seemed somewhat better than that of Mr. Brown.

This Chapter will be concluded in the September issue of Geotechnical News.


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Santiago, Chile
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CONTACT US
conetec.com
Western North America t: 1-800-567-7969 e: insitu@conetec.com
Eastern North America t: 1-800-504-1116 e: insitu@conetec.com