Online

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Volume 34 • Number 3 • September 2016

GEOTECHNICAL news

Field demo: Sand column grouting tests with different types of Microfine cement.

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)

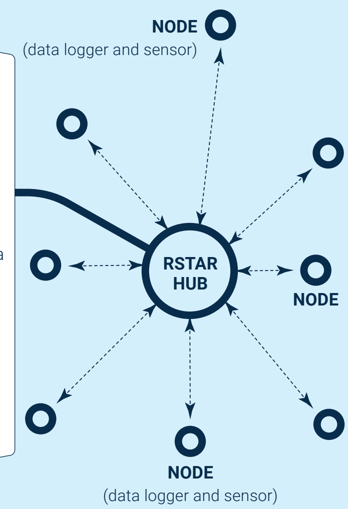
An RSTAR System uses data loggers (nodes) at the sensor level, deployed in a star topology from an active RSTAR Hub containing an RST flexDAQ Data Logger.

rstar
ARRAY RADIO SERIES

FEATURES

- Up to 10 years of battery life from 1 lithium '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 , 868 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 <<<

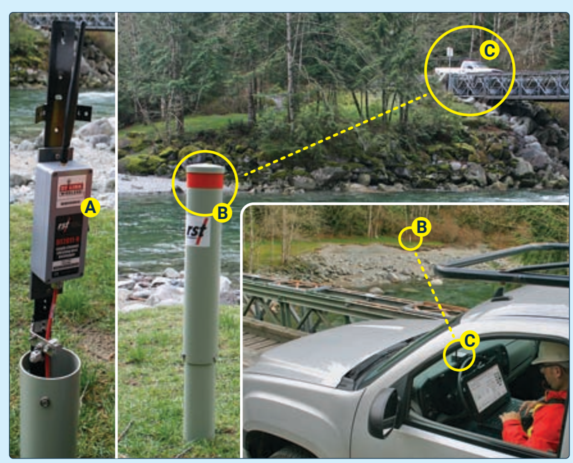
SEMI-AUTOMATED COLLECTION (ON-SITE)

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 '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.

DT SERIES
DATA LOGGERS

RST's "DT Series" Data Loggers accommodate the RSTAR and DT LINK WIRELESS Systems. Compatible sensor types include:
Vibrating Wire, Potentiometers, MEMS Tilt Sensors, Strain Gauge (full bridge) Sensors, Digitally Bussed Sensors, 4-20 mA Sensors, and Thermistors.

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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

ISO 9001

RST Instruments Ltd., 11545 Kingston St., Maple Ridge, BC Canada V2X 0Z5

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: $60.00 CDN per year
- overseas: $95.00 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

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Message from the President

Doug VanDine, President of Canadian Geotechnical Society

I hope you all had a good summer and were able to take some time off and spend it with family and friends.

In the last couple of President's Messages, I have reminisced a bit. I've been surprised at the positive feedback that I've received from both the younger and older CGS cohorts alike. Let me have one more kick at this reminiscing can.

When I started as a geotechnical consultant in the early 1970s, reports were written in long-hand, then put into a "typing pool". Eventually a typed draft came back for your review. Similarly, you sketched rough drafts of the accompanying drawings, submitted them to a draughting department and several days later, voilà, hand-draughted drawings came back for review. After appropriate review, changes were submitted to the typist and draughts-person. A couple more days, and perhaps another iteration, you finalized your report and mailed it (remember Canada Post!), to your client.

Some might say this process, which could take up to a week for a small project, was inefficient; however, in many ways it wasn't. For one, before you started preparing your report, this process required you to have a good idea of what you were going to say and how you were going to say it. The time between the first draft and the final report allowed you time to continue thinking about the project, mull over the findings, your analyses, conclusions and recommendations, then allowed time for an appropriate review by a senior colleague. This process also forced you to plan ahead; there was little room for last minute report preparation. I remember a poster in the

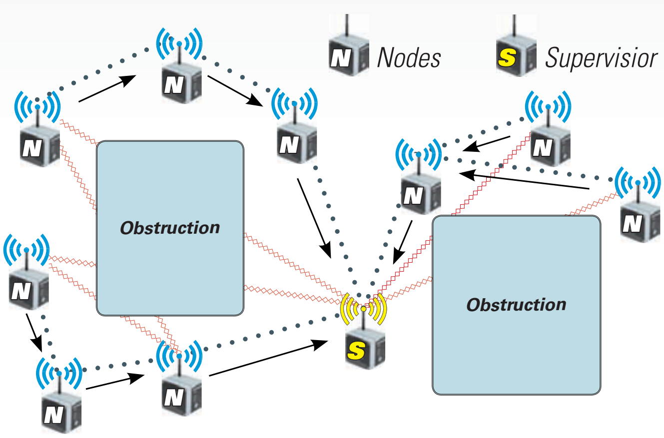
GeoNet Wireless Network

GeoNet is a battery powered wireless data acquisition network compatible with all of Geokon's vibrating wire sensors. It uses a cluster tree topology to aggregate data from the entire network to a single device - the network supervisor. GeoNet is especially beneficial for projects where a wired infrastructure would be prohibitively expensive and difficult to employ.

The network consists of a Supervisor Node and up to 100 Sensor Nodes. Data collected at each node is transmitted to the supervisor. Once there, it can be accessed locally via PC or connected to network devices such as cellular modems for remote connectivity from practically any location.

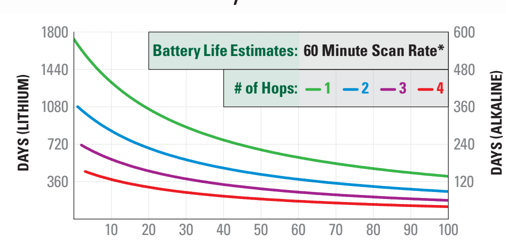
GeoNet Wireless network is self healing and will reconfigure itself to tolerate disturbances to the physical environment.

This topology is more flexible than star networks because it allows data communication to be established over longer distances and around obstructions.



Features & Advantages…

- GeoNet Nodes are comparable in price to a single channel datalogger.
- Uses worldwide 2.4 GHz ISM band.
- Self configuring, easy installation.
- GeoNet will automatically route data around obstructions.
- Nodes separated from network will continue to collect and store data autonomously.
- When network connectivity is re-established the data collected while offline will be transmitted to the supervisor.
- All data collected and sent to the supervisor is also stored on each respective node.
- Long battery life. Most applications measured in years.



*Environmental factors also effect battery life

GEOKON The World Leader in Vibrating Wire Technology™
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info@geokon.com
www.geokon.com

draughting department that reminded us - “A lack of planning on your part doesn’t constitute an emergency on my part.”

Today, you finish the technical aspects of a project one day and for a small project at least, just you on your trusty computer write the report, prepare the drawings and email the final report to your client either later that day or the next. Efficient, yes; a better geotechnical report, I’m not sure. I’ve read lots of geotechnical reports in the last few years that were prepared this way. Many of them read as if more thought and review should have been put into them, both technically and with more regard to good, clear, presentation and communication.

Therefore, the take away thought before you sit in front of your computer to start preparing a report (or a paper, email or any other form of communication), take a little more time to think about the findings, your analyses, conclusions and recommendations. Then consider what is the best way to clearly communicate those to your client (or readers).

What’s coming up this fall? If you aren’t already registered, remember the **69th CGS Annual Conference** is being held in Vancouver from October 2 to 5 (GeoVancouver 2016) (http://www.geovancouver2016.com/). The local organizing committee (LOC), led by co-chairs **Mustapha Zergoun** and **Andrea Lougheed,** have put together a superb conference this year including workshops, field trips, keynote speakers, the technical program, a social program including the Awards Banquet and a partner’s program. I look forward to seeing colleagues from all across the country.

This conference is immediately preceded by the **CGS’s 5th Canadian Young Geotechnical Engineer and Geoscience Conference** (cYGEGC 2016) in Whistler, BC, from September 29 to October 1 (http://cygegc2016.com/index.php/en/welcome/). I know co-chairs **Julian McGreevy** and **Maraika De Groot,** along with their LOC, have also organized great field trips, keynote speakers and a technical and social program for the members under 35 years of age.

Later this fall, **Dr. Ross Boulanger**, Director of the Center for Geotechnical Modeling in the Department of Civil and Environmental Engineering at the University of California, Davis Campus will be criss-crossing the country presenting the **98th CGS Cross Canada Lecture**. This will be a homecoming for Ross, who received his B.A.Sc. in Civil Engineering from the University of British Columbia. As was done for **Dr. Antonio Gens’** Cross Canada lecture in the Spring of 2016, it is hoped that a webinar can be arranged for those CGS Sections that will not host Dr. Boulanger in person.

On a personal note, in late Spring I had the privilege to represent the CGS at the **17th Nordic Geotechnical Meeting** in Reykjavik, Iceland. There I met approximately 300 geotechnical professionals mainly from the northern European countries (but a few CGS members as well). I am very proud and pleased to tell you that the Canadian Geotechnical Society, the **Canadian Geotechnical Journal** and the **Canadian Foundation Engineering Manual** are well known and held in very high regard by many, many geotechnical professionals in that part of the world.

Once again, I would like to thank the CGS staff and all the volunteers who work so hard behind the scenes to keep your CGS running smoothly and keep its reputation so high. A particular thanks to **Don Lewycky**, Editor of the *CGS News* in *Geotechnical News* for the past three years. Don has just agreed to continue as editor for another three years.

If you wish to contact me about this message, or anything related to the CGS, please email me at *President@cgs.ca.*

Provided by Doug VanDine
CGS President – 2015/2016

Message du président

J’espère que vous avez tous passé un bel été et que vous avez pu prendre des vacances et les passer avec la famille et les amis.

Dans les derniers messages du président, j’ai évoqué quelques souvenirs. J’ai été surpris des rétroactions positives de la part des jeunes membres de la SCG ainsi que des membres plus âgés. Permettez-moi de partager un autre souvenir.

Lorsque j’ai commencé ma carrière d’expert-conseil en géotechnique au début des années soixante-dix, les rapports étaient rédigés à la main et envoyés par la suite à un service de dactylographie. Plus tard, vous receviez une ébauche dactylographiée à examiner. De même, les ébauches des dessins qui accompagnaient ces rapports étaient esquissées et ensuite envoyées à un service de dessin et, plusieurs jours après, vos dessins faits à la main vous revenaient pour que vous puissiez les examiner. Après l’examen approprié, les changements étaient envoyés à la secrétaire (dactylographe) et au dessinateur. Quelques jours plus tard, et peut-être après une autre ronde de changements, vous finalisiez votre rapport et l’envoyiez par la poste (par les soins de Postes Canada!) à votre client.

Certains pourraient dire que ce processus, qui pouvait exiger jusqu’à une semaine pour un petit projet, était inefficace. Toutefois, il ne l’était pas à bien des égards. Tout d’abord, avant de commencer à préparer votre rapport, il exigeait d’avoir une bonne idée de son contenu et de sa formulation. La période qui s’écoulait entre la première ébauche et le rapport final vous permettait de continuer à réfléchir au projet, et raffiner les résultats ainsi que vos analyses, conclusions et recommandations. Ceci donnait aussi le temps à un collègue plus expérimenté de l'examiner de manière appropriée. Ce processus vous forçait aussi à planifier; il y avait peu de marge pour la préparation de rapports à la dernière minute. Je me souviens d'une affiche au mur du service de dessin nous rappelant l'adage « Votre manque de planification ne représente pas une urgence pour moi ».

Aujourd'hui, vous donnez la touche finale aux aspects techniques d'un projet en une journée et, pour un petit projet du moins, vous utilisez votre fidèle ordinateur pour rédiger le rapport, préparer les dessins et envoyer le rapport final à votre client plus tard au cours de la journée ou le lendemain. C'est certes efficace, mais je ne suis pas sûr que ceci produise un meilleur rapport géotechnique. Au cours des dernières années, j'ai lu de nombreux rapports géotechniques qui avaient été préparés de cette manière. Plusieurs m'ont semblé pouvoir bénéficier de plus de réflexion et de révision, tant sur le plan technique que pour la présentation et la communication claires et adéquates.

Par conséquent, la leçon à tirer de cela avant de vous asseoir devant votre ordinateur pour commencer à préparer un rapport (ou un article, un courriel ou une autre forme de communication) est de consacrer un peu plus de temps à réfléchir à vos résultats, analyses, conclusions et recommandations. Ensuite, réfléchissez à la meilleure façon pour les communiquer à votre client (ou à vos lecteurs).

Que se passe cet automne? Si vous n'êtes pas déjà inscrit, rappelez-vous que la **69e conférence annuelle de la SCG** a lieu à Vancouver, du 2 au 5 octobre (GéoVancouver 2016) (http://fr.geovancouver2016.com/). Cette année, le comité organisateur local, sous la direction des coprésidents **Mustapha Zergoun** et **Andrea Lougheed**, a préparé une superbe conférence, avec des ateliers, des visites sur le terrain, des invités d'honneur, un programme technique, un programme social qui comprend le banquet de remise des prix, ainsi qu'un programme à l'intention des partenaires. J'ai bien hâte d'y voir des collègues de partout au pays.

Cette conférence est précédée immédiatement par la **5e conférence canadienne des jeunes géotechniciens et géoscientifiques** (cYGEGC 2016) de la SCG qui a lieu à Whistler, en C.-B., du 29 septembre au 1er octobre (http://cygegc2016.com/index.php/fr/welcome/). Je sais que les coprésidents **Julian McGreevy** et **Maraika De Groot,** avec le concours de leur comité local, ont également organisé des visites sur le terrain, des présentations par des invités d'honneur et un programme social et technique à l'intention des membres de moins de 35 ans.

Plus tard cet automne, le **Dr Ross Boulanger,** directeur du Centre de modélisation géotechnique au département du génie civil et environnemental du Campus Davis de l'Université de la Californie, parcourra le pays à titre de présentateur de la **98e Tournée de conférences transcanadiennes de la SCG**. Pour Ross, qui a obtenu son B.Sc. en génie civil de l'Université de la Colombie-Britannique, cela constitue un retour au pays. Comme ce fut le cas pour la conférence transcanadienne du **Dr Antonio Gens** au printemps de 2016, nous espérons pouvoir organiser un webinaire à l'intention des sections de la SCG qui ne pourront recevoir le Dr Boulanger en personne.

Pour conclure sur une note personnelle, j'ai eu le privilège de représenter la SCG à la **17e réunion géotechnique du Nord** à Reykjavik, en Islande, à la fin du printemps. J'y ai rencontré environ 300 professionnels de la géotechnique, principalement de pays de l'Europe du Nord (mais aussi quelques membres de la SCG). Je suis très fier et heureux de vous dire que la Société canadienne de géotechnique, la **Revue canadienne de géotechnique** et le **Manuel canadien d'ingénierie des fondations** sont bien connus et fort estimés par un grand nombre de professionnels de la géotechnique dans cette partie du monde.

À nouveau, j'aimerais remercier le personnel et les bénévoles de la SCG qui travaillent assidûment dans les coulisses, pour assurer le bon fonctionnement de la SCG et maintenir sa bonne réputation. J'adresse des remerciements particuliers à **Don Lewycky**, qui a été rédacteur du bulletin *CGS News* dans *Geotechnical News* pendant les trois dernières années. Don vient tout juste d'accepter un autre mandat de trois ans à titre de rédacteur.

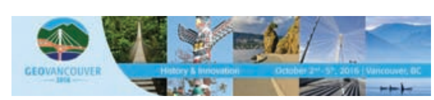
Si vous souhaitez communiquer avec moi au sujet de ce message, ou sur quoi que ce soit en relation avec la SCG, veuillez m'envoyer un courriel à *President@cgs.ca.*

De la part de Doug VanDine
Président de la SCG — 2015-2016

From the Society

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*

Young Geotechnical Engineer Nominations for iYGEC6 Sponsorship Required by October 15, 2016

The CGS and the Canadian Foundation for Geotechnique will sponsor two distinguished 'young geotechnical engineers' to attend the **6th International Young Geotechnical Engineering Conference** (iYGEC6) *http://www.icsmge2017.org/iYGEC/iygec_01.asp* and the first two days of the **19th International Conference on Soil Mechanics and Geotechnical Engineering** (19ICSMGE) *http://www.icsmge2017.org/*. Both are being held in Seoul, Korea in September 2017.

Travel and registration costs, up to $3,500 for each individual, will be provided by the CGS and the Foundation. The two selected individuals will be required to present a paper at the iYGEC6, and to submit an accompanying written paper by March, 2017.

Any Canadian (or permanent resident), active 2016 CGS Members or Student Members born after January 1, 1982 are eligible. Nominations can be made by individuals, their employers or faculty supervisors. The nomination package, including a 250-word abstract, should not exceed 3 letter-size pages (Times Roman size-12 font). **Nominations must be received by CGS Headquarters**, *admin@cgs.ca*, by **October 15, 2016**.

Selection will be based on:

1. Leadership/activity in the Canadian geotechnical community (i.e., participation in the CGS, organization of conferences/special events, university student organizations, etc.)
2. Contribution to practice (consulting experience, technology transfer)
3. Publications/presentations (journal papers, conference papers, patents, workshops, etc.)
4. In the case of students, academic standing in graduate courses
5. Submission of an appropriate 250-word abstract (in English or French) of the proposed presentation (and paper) on any topic related to geotechnical engineering, geological engineering or geoenvironmental engineering or geoscience.

Nominees not selected for the CGS/Canadian Foundation for Geotechnique sponsorship are encouraged to permit CGS to submit their abstracts to the iYGEC6 – this should be indicated in the nomination package. Non-sponsored individuals will have to cover their own costs to attend the conference.

If you have any questions, contact **Lisa McJunkin**, CGS Director of Communications and Member Services, *admin@cgs.ca*, 604 277 7527 or 800 710 9867.

Les candidatures de jeunes géotechniciens pour la commandite de l'iYGEC6 doivent être soumises d'ici le 15 octobre 2016

La SCG et la Fondation canadienne de géotechnique commanditeront deux jeunes géotechniciens distingués pour qu'ils assistent à la **6th International Young Geotechnical Engineering Conference** (iYGEC6) *http://www.icsmge2017.org/iYGEC/iygec_01.asp* et aux deux premiers jours de la **19th International Conference on Soil Mechanics and Geotechnical Engineering** (19ICSMGE), *http://www.icsmge2017.org/*. Ces deux conférences ont lieu à Séoul, en Corée, en septembre 2017.

Les frais de déplacement et d'inscription, allant jusqu'à 3 500 $ par personne, seront assumés par la SCG et la Fondation. Les deux personnes sélectionnées devront présenter un article à l'iYGEC6 et soumettre un article écrit l'accompagnant d'ici mars 2017.

Tout membre actif ou membre étudiant canadien (ou détenant la résidence permanente au Canada) de la SCG en 2016 né après le 1er janvier 1982 est admissible. Les candidatures peuvent être soumises par les candidats, leurs employeurs ou directeurs de travaux à l'université. Le dossier de candidature, incluant un résumé de 250 mots, ne devrait pas dépasser trois pages de format lettre (police Times Roman, taille 12). **Les candidatures doivent être reçues au siège social de la SCG, à l'adresse admin@cgs.ca, d'ici le 15 octobre 2016**.

La sélection sera basée sur :

1. Le leadership/la participation dans la communauté géotechnique canadienne (c.-à-d., la participation à des activités de la SCG, la contribution à l'organisation de conférences/d'événements spéciaux et à des organismes étudiants universitaires, etc.)
2. La contribution à la pratique (expérience en consultation, transfert technologique)
3. Les publications/présentations (articles de revue, articles de conférence, brevets, ateliers, etc.)
4. Dans le cas d'étudiants, le rendement académique dans les cours de niveau supérieur
5. La soumission d'un résumé approprié de 250 mots (en anglais ou en français) de la présentation proposée (et de l'article) sur tout sujet lié à la géotechnique, au génie géologique, au génie géoenvironnemental ou à la géoscience.

Les candidats non sélectionnés pour la commandite de la SCG et de la Fondation canadienne de géotechnique sont encouragés à permettre à la SCG de soumettre leurs résumés à l'iYGEC6; cela devrait être indiqué dans leur dossier de candidature. Les personnes non commanditées devront assumer les coûts de leur participation à la conférence.

Si vous avez des questions, communiquez avec **Lisa McJunkin**, directrice des communications et des services aux membres de la SCG, à *admin@cgs.ca*, au 604-277-7527 ou au 1-800-710-9867.

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 **December 30, 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 Local Sections of the Canadian Geotechnical Society

The Heritage Committee believes that the history of the local sections of the Canadian Geotechnical Society to be valuable part of the Society and its members. The CGS Heritage Committee would like to assemble if at all possible, a collection of historical summaries of all the sections. Hopefully every local chapter of the CGS will take the time to gather their archives and write their own history.

Please contact the Chair of the CGS Heritage Committee, Dr. **David Cruden**, at *dcruden@ualberta.ca* if you have any questions.

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 87th episode of GIN. Only one article this time, and that's by the editor!

To all of you 'out there' – if you believe that GIN has any value, please help by contributing. Surely you have practical experiences or lessons you've learned that are worth sharing. I hope for some 200-300 word abstracts.

If you believe that GIN has any value, PLEASE help by sending me an abstract

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

The previous three GINs included articles about instrumentation for braced excavations, embankments on soft ground and cut slopes and landslides in soil and in rock. Here's one about driven piles and bored piles (drilled shafts).

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

In the previous GIN I asked if anyone would be willing to write an article on monitoring of embankment dams, but you've been silent. So I'll ask the question another way – do you know of anyone who might be willing to write an article? If yes, will you please let me have contact information?

Third International Course on Geotechnical and Structural Monitoring, June 2016 in Italy

The Third International Course on Geotechnical and Structural Monitoring is now history. For the first three editions of the course (held in 2014, 2015 and 2016), more than 330 people from 48 countries joined us. In addition, 42 companies provided exhibits.

Here are two photos from this year's course. For more, take a look at the photo gallery on *www.geotechnicalmonitoring.com/en/july-newsletter.*

For the 2017 edition we're looking at two options for location: again in the beautiful small Tuscany town of Poppi, or perhaps in Rome. Watch this space! We're planning to add some side courses on the day before the main course to provide practical basic know-how on how to use the most common monitoring systems.

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*

Amor, pesetas y el tiempo para gozarlos ("Love, money and the time to enjoy them") - Spain

2016 course registrants on balconies in the castle.

Unforgettable street party.

General role of instrumentation, and summaries of instruments that can be considered for helping to provide answers to possible geotechnical questions. Part 4.

John Dunnicliff

Introduction

This is the fourth 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. Part 3, in June 2016 GIN, covered cut slopes and landslides in soil and in rock. This Part 4 is about driven piles and bored piles (also called drilled shafts).

Four introductory points were made in December 2015 GIN (www.geotechnicalnews.com), for Part 1 of this series of articles, and these also apply here.

Driven piles

General role of instrumentation

The subsurface length of a driven pile cannot usually be inspected after driving; thus, its physical condition and alignment are unknown. Subsurface geotechnical conditions are rarely known with certainty, and therefore the design of driven piles involves assumptions and uncertainties that are often addressed by conducting instrumented full-scale tests. Tests may examine the behaviour of the pile under load applied to the pile head or under load caused by settlement of soil with respect to the pile.

Defects in piles can be created during driving, and inspection procedures are available for examining the condition and alignment after driving. Certain types of driven pile cause large displacements and changes of pore water pressure in the surrounding soil, and these may in turn have a detrimental effect on neighboring piles or on the stability of the site as a whole. Instrumentation can be used to quantify the consequences of pile driving and thus to assist in planning any necessary action.

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

Table 8 lists the possible geotechnical questions that may lead to the use of instrumentation for driven piles, together with possible instruments that can be considered for helping to provide answers to those questions.

Table 8. Some instruments that can be considered for monitoring driven piles

Possible geotechnical questions	Measurement	Some instruments that can be considered
What is the load-movement relationship of the pile	Displacement at head	Dial indicators with reference beams Wire/mirror/scale Surveying methods Remote methods
	Load at head	Load cell
	Displacement at toe	Telltales
	Stress along pile	Embedment or surface-mounted strain gauges (Fibre-optic instruments)
Has the capacity of the pile been reduced by defects caused during driving?	Curvature of pile	Inclinometer
	Condition of pile	Integrity testing

Bored piles (drilled shafts)

General role of instrumentation

Many uncertainties exist during design of bored piles (also called drilled shafts), and instrumentation plays a role in determining the load-movement relationship, by conducting load tests. Concrete integrity is often uncertain during construction, particularly when they are constructed in granular soils below the water table or in softer, squeezing clays, when concrete slump is inadequate, or when concrete placement practices are inferior. Instrumentation can be used to examine the integrity of the concrete. For piles cast under support fluid, concrete integrity at the pile tip is particularly important.

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

Table 9 lists the possible geotechnical questions that may lead to the use of instrumentation for bored piles, together with possible instruments that can be considered for helping to provide answers to those questions.

Table 9. Some instruments that can be considered for monitoring bored piles

Possible geotechnical questions	Measurement	Some instruments that can be considered
What is the load-movement relationship of the pile?	As in Table 8 Load at toe or in pile	As in Table 8 Osterberg load cell
What is the integrity of the concrete?	Condition of pile	Integrity testing

Final closure and post closure care of landfill cells: Lessons learned by one municipality in Alberta

Carol A. Kehoe

Background

The Canadian Public Sector Accounting Board (PSAB) has made recommendations concerning how municipalities should account for the long-term financial costs of managing the liabilities associated with closure and post-closure care of solid-waste landfill sites under PS 3270 (Financial Reporting & Assurance Standards Canada, n.d.). Closure activities to be accounted for include final cover and storm water management, monitoring of leachate, water quality, landfill gas, and gas recovery. After final closure, financial reporting of post closure care activities should include ongoing treatment and monitoring of leachate and landfill gas, monitoring of surface water and groundwater, and ongoing maintenance for the full post-closure care period. Information on the environmental condition of municipal landfills is needed to account for these costs.

The City of Calgary currently owns and operates three solid waste landfills, and has closed an additional six solid waste landfills over the years. East Calgary, Shepard and Spyhill solid waste landfills were originally permitted under the Provincial Board of Health and have been receiving wastes for close to 50 years. Alberta Environment became responsible for regulating landfills throughout Alberta effective September 1996 (Rush, 1996). New operating approvals, issued between 2001 and 2003, cancelled the old Board of Health permits. Blackfoot, Highfield, Manchester Yards, Nose Creek, Ogden and Springbank were closed during the time when solid waste landfills were under the jurisdiction of the Provincial Board of Health.

The City of Calgary has made financial provision for environmental monitoring and maintenance of its closed landfills since 1990 (City of Calgary, 1990) and has completed numerous intrusive investigations and remediation activities over the years.

Assessing the environmental condition of municipal landfills

In 2007, The City of Calgary initiated a program to re-assess its landfills, coinciding with The Province of Alberta introducing new guidelines for the remediation of contaminated sites: Alberta Tier I Soil and Groundwater Remediation Guidelines and Alberta Tier 2 Soil and Groundwater Remediation Guidelines. Approved facilities including solid waste landfills holding an Environmental Protection and Enhancement Act (EPEA) approval were to adopt the new guidelines as outlined in their individual operating approval (Province of Alberta, 2007).

The program goals were to provide clarity regarding the environmental requirements for the landfills, identify potential environmental impacts at the landfills, provide clarity regarding the financial costs of ongoing landfill management, and to manage the program well through the engagement of internal and external stakeholders and by meeting corporate records management requirements.

The key deliverables for the program were:

- Historical reviews
- Data gap analyses
- Preliminary conceptual site models
- Intrusive investigations
- Human health and ecological screening level risk assessments
- Methane gas surface emissions surveys
- Updated conceptual site models

Achieving the goals of the program provided addition information on the landfills that was useful in forecasting the long term financial costs associated with closure and post-closure care of landfills. In 2007, those costs were \$10.9M (City of Calgary, 2009); in the 2015 Annual Report those costs were \$87.5 million (City of Calgary, 2015).

Lessons learned

The program resulted in more extensive groundwater, leachate and landfill gas monitoring networks to assess potential environmental impacts; improvements in landfill gas controls; improved relationships with both internal and external stakeholders; and, improved documentation of municipal processes related to historic landfill development.

Similar programs have been undertaken by other municipalities, including but not limited to the cities of Hamilton, Ottawa, and Toronto (Davis, 2011; Geddes, 2004; Griffiths, 2011). Programs identified both the need for capital investment to improve the environmental performance of the closed landfills, and increased operating budget to meet regulatory requirements for environmental monitoring, maintenance and recovery.

Early in The City of Calgary's program, it was decided that common assumptions made in past investigations and remedial activities should be confirmed as part of this program. Challenging those assumptions provided some of the more interesting lessons learned.

Intrusive investigations begin in the file room

Historically, investigations were undertaken with a targeted scope of work developed for a specific project. The goals of this program were broader than previous studies and included a detailed historical review and gap analysis undertaken prior to scoping the intrusive investigation.

A municipal corporation such as The City of Calgary is a complex organization. Development and delivery of municipal services is a shared responsibility across departments to balance multiple municipal needs. In Calgary, this balancing of municipal needs over time has resulted in portions of landfill lands sold, subdivided, converted to roads and interchanges; leased and developed to accommodate communication towers, commercial tenants and public recreation; and, used to accommodate buried and overhead infrastructure. Significant time was spent with other internal business units looking for corporate records that pertained to the landfills. Relevant records, including geotechnical and hydrogeological investigations, annual reports, regulatory permits, access and lease agreements, development plans, waivers, block profiles, road construction and road closures, and historic soil and water test results were found across the corporation, including Corporate Archives. Not surprisingly, some pertinent documents were also found in filing cabinets and storage boxes that had not yet been classified or added to the corporate record.

The historical review included a project to digitize the physical documents including site plans, reports, drawings and correspondence. The electronic documents are easily accessible and searchable from an electronic file system. After the documents were digitized, the original records were transferred to secure storage for retention, as required under the operating approvals and in accordance to the corporate records management policy. It is anticipated that the electronic catalogue will support improved knowledge transfer to future projects.

A fence is just a fence

Home owners typically install a fence along a property line to clearly separate their land from their neighbors. A fence is therefore commonly but erroneously interpreted as a property line.

When completing this program, it was found that fences at the landfills were typically not installed on the legal property boundary. Security fences were installed at East Calgary, Shepard and Spyhill along the perimeter of landfill operations and around more sensitive infrastructure such as landfill gas facilities. Fences were used to define leased areas, to demarcate parking areas, and to limit illegal dumping and trespass. Fences intended to be a physical barrier were sometimes misinterpreted as property lines, so that past investigations, methane surveys and site inspections sometimes stopped at those fence lines instead of considering the entire landfilled area.

Maps of the landfills have now been produced that clearly show both the property boundaries and the locations of fences, reducing the ongoing risk of a fence line being assumed to be a property line.

A road is a road is a road

Three distinct types of roads were identified during the program.

A few functional roads were found to be outside of legal road plans. It was only when looking for additional lines of evidence, including road plans and construction drawings, that it became apparent that some of the roads was not roads within a road plan.

Several road plans were identified within landfills that were still open for future public roads, including some within the operating landfills. Such future roads would be contrary to the long term development plans for those sites.

Lastly, several roads were identified that had been constructed after a landfill was closed. New road plans were filed and the landfill legal land description was also changed.

Understanding the different types of roads was important when considering if any of the intrusive investigations needed to include roads, and when considering what authorization was needed to investigate along roads. A follow-up project was undertaken to update legal plans as appropriate for the individual landfills.

Stormwater ponds

Ongoing requirements related to landfill stormwater ponds varied depending on where and when the stormwater ponds were constructed.

Stormwater ponds constructed on the operating landfills were approved under each individual provincial approval to operate. Ponds included both evaporation ponds within the sites and ponds designed to hold water for testing prior to release into the watershed.

Stormwater ponds within Springbank and Blackfoot were constructed after both sites were closed. Although each landfill operated and closed under a Provincial Board of Health permit, the ponds and final cover design were approved under an amendment to The City of Calgary's approval for municipal stormwater management. Both ponds were designed to manage on-site overland flows for release to a stormwater catch basin. There was no requirement to hold and test the water prior to release.

The stormwater pond within Ogden was part of the final closure plan approved by The Province when the site was operated and closed under a Provincial Board of Health permit. That pond is an evaporation pond with no release to the watershed.

A review of the stormwater pond construction and authorizations provided clarity on their environmental requirements and details necessary for planning the intrusive investigation.

Buried waste may not be a municipal landfill

The program included a detailed review of available municipal records dating back to the first public scavenger hired in 1885. Annual engineering reports and historical drawings were particularly valuable in documenting the locations and disposition of municipal wastes and rubble over the years. From this work, The City of Calgary has increased confidence in knowing the location and operating periods of closed landfills managed by the municipality.

Buried waste has been encountered at locations other than Calgary's known municipal landfills. Under historic provincial regulations land owners were allowed to dispose of their own rubble and inert waste on their property, and to fill in low lying areas with inert waste and rubble. (Province of Alberta, 1936). Also, like many cities, Calgary has grown through a series of annexations (City of Calgary, 2016) including the towns of Forest Lawn, Bowness, Montgomery, and Midnapore, and rural lands annexed from both Rockyview County and the Municipal District of Foothills.

Conclusion

A program to re-assess The City of Calgary's landfills provided clarity regarding the environmental requirements and financial costs of ongoing landfill management. It resulted in a more extensive groundwater, leachate and landfill gas monitoring networks to assess potential environmental impacts, and a searchable electronic catalogue of records to support knowledge transfer to future projects.

Given that municipalities must account for the long term financial costs associated with closure and post-closure care of solid waste landfills, more studies will likely be undertaken by other municipalities to better document the environmental condition of those sites. When undertaking such studies it may be useful to question what proof of evidence exists for any assumptions made when scoping investigations. Establishing solid lines of evidence to test such assumptions will support the development of a comprehensive conceptual site model.

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Carol Kehoe, M.Sc., P.Eng.
The City of Calgary
Waste & Recycling Services
PO Box 2100, Station M, #27
Calgary, Alberta T2P 2M5
E-mail: carol.kehoe@calgary.ca.

Paolo Gazzarrini

Overture

44th episode of the Grout Line and with all the grouters busy in preparing abstracts for Grout 2017 (Hawaii-July 2017) I don't have the usual participation in the Grout Line submissions. There were more than 200 abstracts submitted for the conference!

For this issue, only a short report from Prof. Scott Kieffer (course director) about the 37th Grouting Fundamentals Course.

The 37th annual Grouting Fundamentals short course just concluded at UT Austin with great success!

Since 1979 the Grouting Fundamentals course (previously hosted by the Colorado School of Mines, University of Florida, and Missouri-Rolla) has covered pressure grouting as a method to improve geotechnical characteristics of soils and rock masses. A broad range of grouting procedures and applications is addressed, including compaction, permeation, and rock fracture grouting, design and construction of grout curtains, cut-off walls and composite seepage barriers, deep soil mixing, and grouting of high capacity ground anchors, soil nails, and micropiles.

The 1/2-day field demonstration is a key aspect of the course, where contractors, manufacturers and suppliers converge to provide hands-on experience with grouting materials and procedures. The Demo included a variety of chemical and cementitious grout materials, high shear grout plants, anchor grouting and load testing, microfine grout penetrability tests, slab lifting with polyurethane grouts, and a live display of compaction grouting.

From all accounts the recent course at UT Austin was amongst the best installments in the course's long and rich history. **For details regarding the May 15-19, 2017 course visit**: ***www.groutingfundamentals.com***

Only a short comment of mine. I think Scott forgot to mention the excellent Texan BBQs we had during the week!

Field Demo: June 13 – 17, 2016
Grouting Fundamentals & Current Practice at UT Austin.

Field Demo: Sand column grouting tests with different types of Microfine cement.

Field Demo: Annulus grouting.

Not only grouting!

Geotextile filter case study: Alouette Dam spillway rehabilitation British Columbia, Canada

Jonathan Fannin, Editor

Professor of Civil Engineering, University of British Columbia

Jonathan Fannin

Readers of this column may recall that, in the GN: June 2015 issue, I reported on filter applications with a return to geotextile "basics" that examined the measurement and reporting of pore size opening, cross-plane hydraulic conductivity (permittivity), tensile strength and soil-geotextile compatibility. In projects where the cost of remedial works is anticipated to be significant, I acknowledged that the state-of-practice is first to identify a candidate geotextile on the basis of index tests for pore size, permittivity and strength, and then to evaluate its suitability for the proposed construction application from laboratory compatibility testing of a sample of the base soil in combination with the candidate geotextile filter. In this current article, I describe a case study that illustrates the approach with reference to the specification of a geotextile used in rehabilitation of the drainage system beneath the spillway of the Alouette Dam in Canada.

Alouette Dam, British Columbia

The Alouette Dam forms a part of the Alouette-Stave Falls-Ruskin Hydroelectric Complex, a sequence of three dams that is located about 65 km east of Vancouver, Canada. It is operated by BC Hydro, a Crown corporation that supplies most of the electrical energy for the Province of British Columbia. BC Hydro operates more than 70 dams, with maximum heights up to 240 m, and the continued safety and operation of those dams is an integral part of the Provincial economy.

The Alouette Dam controls the level of Alouette Lake. Impounded water is diverted through a tunnel near the head of the lake to a powerhouse and hence into Stave Lake, from where it passes sequentially through the Stave Falls powerhouse into Ruskin Lake and, lastly, through the Ruskin powerhouse. In total, the three powerhouses have the capacity to produce 205 MW of electricity. The Stave Falls dam was the first of the sequence to be constructed. It is a concrete-gravity and rock-fill dam, approximately 120m long, that was completed in 1912 and subsequently raised to a height of approximately 24 m in 1925. The Alouette Dam is an earthfill dam, approximately 290 m long and 20 m high, completed in 1928. The Ruskin Dam is an overflow concrete gravity structure, approximately 110 m long and 60 m high, completed in 1930.

The Alouette Dam is located on a glacial outwash formation. The earthfill structure incorporates a cut-off trench, a low level outlet and a spillway. The majority of material used in construction of the dam was obtained from excavation of soil for the adjacent spillway. The construction method involved excavation by steam-shovel and placement by dumping from railway wagons at or near the outer slopes of the dam, followed by washing toward a central pool in the embankment in a manner of "semi-hydraulic filling" (Carpenter, 1927) for which "all the material in the core area was thus deposited under water and the voids in the coarser materials forming the slopes at the edges of the core were well filled with fines". In 1983, the need for a seismic upgrade of the structure to accommodate revised design seismic loading resulted in the construction of a new earthfill dam located immediately downstream of the original hydraulic fill dam (Hartford and Lou, 1994).

Spillway structure rehabilitation

The original concrete-lined spillway of the Alouette Dam was built in 1926, with control gates and an overflow weir, in order to carry flood water around the earthfill dam and safely discharge it below the downstream toe. The elevation drop between the spillway crest and the Alouette River was accommodated by an ogee structure at the end of the spillway: it was founded directly on the excavated ground and incorporated a concrete cut-off toe wall on all three sides. Erosion protection was afforded by submerging the discharge section at the end of the ogee structure, thereby inducing a hydraulic jump in the backwater pool, which acts to dissipate energy from the flow of water (Carpenter, 1927). Further erosion protection was afforded by a concrete slab that extended beyond the toe wall, and by a transition blanket of riprap along the rise of the channel through which discharge over the spillway was returned to the Alouette River. The spillway was partially rebuilt in 1955 following flood damage (Brown and Nielson, 1992), and further improved in 1961, 1983 and 1985 before undergoing a major rehabilitation in 1992 (see Fig. 1).

The “Alouette Dam – 1992 Spillway Rehabilitation Project” was undertaken to accommodate a revision to the design extreme flood magnitude.

Figure 1. General view.

Figure 1b. Inclined sidewall of the spillway channel.

Additionally, it was undertaken to address some observed deficiencies that had been identified through engineering inspection, including erosion and undermining of the spillway foundation, as well as deterioration of the original concrete. Erosion beneath the spillway was attributed to no provision being made, at the time of original construction, for under-drainage between the spillway and its foundation. The rehabilitation works included placing a new spillway gate, replacing the original portions of the spillway channel, and constructing a new stilling basin. Importantly, the rehabilitation work included filter and drainage provisions underneath the spillway channel that were intended to prevent uplift of the slabs during major floods due to potential large fluctuations in water pressure during flood routing.

The safe, long-term operation of the spillway is governed by the capacity of the under-drain to collect, depressurize and remove any groundwater that seeps into it from the foundation soil. Further, it must collect and remove any channel flow that enters it through the concrete liner during a period of spillway use. In order to provide for this design function, the drainage system must be protected against ingress of the foundation soil on which it rests. Accordingly, a filter was placed between the foundation soil and drainage layer.

The configuration of the drainage system differs along the length of the spillway. Where possible, such as the spillway invert, granular filters and drains were used. A total of three combinations of materials were used in construction: (i) a granular filter and granular drain, (ii) a geotextile filter and granular drain, and (iii) a geotextile filter and a geosynthetic drain. More specifically, a granular filter and drain was specified in contact with the base of the spillway channel where the slope is relatively flat. A combination of geotextile filter and granular drain was used on the sloping section of the spillway base. On the steeply inclined

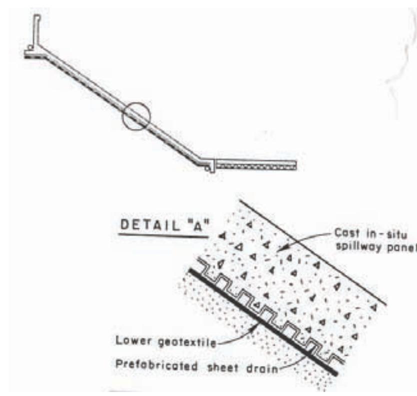

Figure 2. Configuration of the geotextile filter and sheet drain.

sidewalls of the spillway channel, the specification required a combination of geotextile filter overlain by a moulded plastic sheet-drain of single cuspate construction (see Fig. 2). The resulting geocomposite filter-drain was custom-designed and assembled on site: several off-the-shelf pre-assembled products were available at the time, but were deemed unsuitable for this application. The sidewalls were excavated at an ngle of approximately 1.5H:1V over a length of 170 m on one side, and 50 m on the other side, of the spillway channel. The combination of geosynthetic filter and drain was specified for the sidewalls because it was expected the steep cut would present a significant challenge for economic placement, and satisfactory compaction, of a granular material filter and drain. Further, there was concern that heavy rainfall might yield erosion of filter sand on the steep sidewalls and result in construction delays.

Specification of the geotextile filter

The original spillway lining was founded directly on a sequence of sand and silts of the Fort Langley Formation. Drilling in the period 1979 to 1990 established the geologic sequence to consist of very dense outwash sands overlain by inter-layered fine sands and silts which, in turn, are overlain by stiff over-consolidated clayey silts. No bedrock was encountered within 100 m of ground level. Additional drilling in 1991 included two mud-rotary drill-holes on the right bank, three auger-holes through the spillway forebay and ogee areas, and seven large diameter churn-holes (wells) that were drilled around the plunge pool area. The strength of the silts and inter-layered sands and silts along the alignment of the spillway varies from firm to very stiff.

The primary function of the geotextile is filtration. It must protect against any unacceptable movement of base soil through it and into the void space of the drain, without adversely impeding groundwater seepage across it and into the drain. Grain size analysis on samples retrieved from boreholes indicated the base soil to be sandy silt, with a mean grain size $0.01 \leq d_{50} \leq 0.05$ mm, a value $0.04 \leq d_{85} \leq 0.6$ mm and a coefficient of uniformity $C_U \approx 20$. The results of Atterberg limits testing classified the base soil as low plasticity silt (ML).

The under-drain below the sidewall of the spillway is required to handle inflow from two separate sources:

- groundwater seepage from the base soil; and,
- leakage through joints or cracks in the concrete liner during spillway operation.

For purposes of design, it was assumed the geotextile filter is subject to steady unidirectional flow from the base soil into the drain. Three candidate geotextiles were selected in order to evaluate filtration compatibility with the base soil. They were all needle-punched nonwoven geotextiles. The preliminary selection was made on the basis of values for pore-size opening, cross-plane permeability and strength reported in technical literature by the manufacturers.

> **"A significant finding of the filtration compatibility testing that was common to the HCR results, and also to the GR results, was the absence of any continuous or significant piping..."**

At the request of BC Hydro, the characteristic value of pore size opening was independently verified for the candidate geotextiles with reference to the Filtration Opening Size (FOS), a hydrodynamic sieve test. The index test was performed by a commercial laboratory in Canada, on samples of candidate geotextile provided to BC Hydro by the respective manufacturers. A similar value of $O_{95} \approx 0.07$ mm was determined for each of the three geotextiles. Filtration compatibility of three candidate geotextiles was then evaluated through program of Gradient Ratio (GR) testing (after ASTM D 5101) and Hydraulic Conductivity Ratio (HCR) testing (after Williams and Abouzakhm, 1988; now standardized as ASTM D 5567). The testing was performed by a commercial laboratory in the USA, using samples of the base soil taken from site in combination with the candidate geotextiles.

The base soil against which the geotextile is placed is non-plastic silt. Given the characteristic grain size of the base soil ($0.04 \leq d_{85} \leq 0.6$ mm), and given the index pore size opening of the candidate geotextiles ($O_{95} \approx 0.07$ mm), the capacity for soil retention was identified as very important to a confident evaluation of soil-geotextile compatibility. Analysis of the filtration compatibility test data placed considerable emphasis on the relation between hydraulic conductivity of the soil-geotextile composite zone over the duration of the test. More specifically, interpretation of compatibility was based on the variation of hydraulic conductivity with elapsed time, and also its variation with pore volume exchange across the geotextile. A significant finding of the filtration compatibility testing that was common to the HCR results, and also to the GR results, was the absence of any continuous or significant piping of soil from the reconstituted test specimen through the geotextile. Furthermore, all three candidate geotextiles yielded a very similar performance. The original development of the HCR test was intended for soil with a hydraulic conductivity less than or equal to 5×10^{-2} cm/s. The hydraulic conductivity of the non-plastic silt at the Alouette Dam was found to be considerably lower than this value. Accordingly, greater emphasis was placed on the HCR test results, when evaluating the compatibility of the nonwoven geotextile and silt for purposes of design.

Results of the laboratory testing were used to inform the selection of material properties reported in the specification documents of the contract for the project. The geotextile was required to have an opening size $40\ \mu m \leq O_{95} \leq 75\ \mu m$. Requirements for material strength, permittivity and UV degradation were established with reference to routine design guidance at that time. From comparison to current practice, they would be in general conformance with an AASHTO Class 2 material. Details of the three geotextiles that were tested and found acceptable to B.C. Hydro were reported in an appendix to the specification document.

Prior to ordering the geotextile, the Contractor was required to submit for acceptance, a 1m x 1m sample of the proposed material, including the manufacturer's name and product name. The transportation, storage and installation of the geosynthetics was specified in a manner that limited the maximum total duration of exposure to ultraviolet light to a period not exceeding 14 days. Prior to installation of the geotextile, the Contractor was required to submit for review and acceptance, information on the manufacturer, product name, lot number and roll number of each geotextile roll delivered to the site. No installation was to occur prior to the acceptance of all submissions.

Construction and performance monitoring

The spillway rehabilitation project commenced in 1992, and was completed over a period of several months. The filter was specified to be placed in a slackened condition such that it would conform to the subgrade surface area. No construction equipment was allowed to operate directly on the geotextile. On the steep side-slopes of the spillway channel, placement of the geosynthetic filter and cuspated sheet-drain was found relatively straightforward. In contrast, and as anticipated in the early stages of design, it proved challenging and time-consuming to achieve compaction requirements for the granular filter and drain on the steeper sections of the spillway profile. With regard to strength of the geotextile, and construction survivability, engineering inspection revealed no physical damage to it during the period of installation.

The laboratory testing of soil-geotextile compatibility had identified three candidate geotextiles, and the Contractor elected to select one of them for use in construction. At the time of writing the specification documents, the laboratory evaluation of soil-geotextile compatibility had been made for a select combination of confining stress and hydraulic loading, and over a relatively short duration of time. Since completion of construction, nearly 25 years ago, the performance of the drainage system has been subject to ongoing performance monitoring. The performance monitoring includes pore water pressure measurements within the drainage system under the stilling basin. Observations from the monitoring program indicate the geosynthetic filter and sheet-drain are part of a composite drainage system whose overall performance, like that of the rehabilitated spillway, is fully in accordance with design expectations (B.C. Hydro, personal communication).

Closing remarks

It is widely-accepted practice to specify a geotextile for a filtration application with reference to (i) provision of adequate material strength and durability, (ii) an empirical rule governing base soil retention, and (iii) an empirical rule governing base soil permeability. The development of current practice is shown to be founded on a long-standing body of field and laboratory experience, acquired in many countries, over a period of more than 50 years – something that we have addressed in earlier GN:Geosynthetics articles.

Soil-geotextile compatibility is predicated on the geotextile having adequate strength to ensure no adverse damage during the process of installation (termed 'construction survivability). Thereafter it must endure the working environment of the installation over the service life of the structure (termed 'durability'). Recommendations for construction survivability of geotextiles were first addressed in a systematic study conducted in 1972. They have since been refined over time, with current practice giving recognition to several classes of material strength, each of which is established with reference to standardized index tests. Durability studies have been ongoing for a similar period of time, with early contributions associated with specific case history records dating back to a 1969 revetment application in Florida and a 1970 dam application in France. Insights to the governing influence of thermal-photo-oxidation degradation mechanisms on material durability are consistent with field observations over many years. Indeed, the basis for provision of adequate material strength over the service life of the structure is now well-understood.

"On the steep side-slopes of the spillway channel, placement of the geosynthetic filter and cuspated sheet-drain was found relatively straightforward."

The requirement for soil-geotextile filtration compatibility is contingent on there being no unacceptable erosion as a consequence of soil loss through the geotextile while, at the same time, providing for unimpeded flow of water from the soil through the geotextile. Empirical design criteria for soil retention and permeability were first established for woven and nonwoven geotextiles in the period 1972-1975. They have since been developed and refined, mostly from the findings of laboratory studies, and with occasional reference to companion theoretical analysis. The criteria relate a characteristic opening size of the geotextile to a characteristic grain size of the soil. The opening size of the geotextile is typically reported with reference to one of three standardized index test methods of sieve analysis, for which laboratory comparisons yield a similar but not identical value of characteristic pore size opening. Accordingly, there are modest differences between the various empirical criteria that are used for soil retention.

Filtration compatibility is evaluated from permeameter testing. Since the first laboratory studies reported in 1972, the configuration of test equipment has evolved to include development of standardized test methods for rigid-wall permeameter testing of geotextile compatibility with a relatively coarse-grained base soil, and flexible-wall permeameter testing of compatibility with fine-grained soil. In the last 20 years, the state-of-the-art has advanced to include refinements to equipment and procedures that have enabled compatibility testing in unidirectional and reversing flow with a mechanics-based interpretation of the results which accounts for the combined influence of hydraulic gradient and effective stress. The findings of these systematic laboratory studies not only provide confidence in the use of empirical design rules, but have also provided a means to quantify the inherent conservatism of the empirical criteria that are used in routine practice. In more critical or severe applications, the standardized tests also provide a means to evaluate soil-geotextile compatibility in advance of construction. In such projects, the state-of-practice is first to identify a candidate geotextile on the basis of the reported values for its strength, opening size and permittivity from index testing, and then to evaluate its suitability for the proposed construction application from performance testing of a sample of the base soil in combination with that candidate geotextile filter.

Acknowledgements

The preparation of this article has drawn upon a number of published and unpublished sources. The permission of BC Hydro Dam Safety, most notably Stephen Rigbey and Kay Ahlfield, to report on information relevant to the selection and specification of a geotextile filter at the Alouette Dam is gratefully appreciated. The specification of the geotextile was developed in close collaboration with the Project Engineer (Stephen Garner, BC Hydro, now retired), with whom it was a pleasure to work, and whose experience was instrumental to the success of the project.

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Tracer tests: Experimental verification of a new predictive equation for effective porosity in stratified alluvial aquifers

Robert P. Chapuis

Abstract

Tracer tests in aquifers involve effective porosity, n_e, and longitudinal dispersivity, α_L, which are known to depend upon the aquifer heterogeneity. There is no reliable method to predict n_e, which is extracted by fitting the test data of a breakthrough curve to a theoretical curve. Chapuis (2015) used physical principles to derive a new predictive equation for n_e in stratified alluvial aquifers. The new equation is verified experimentally in this paper, using laboratory and field data for tracer tests.

Introduction

Large populations depend upon groundwater and wells. Starting in the 1980s, many countries have developed protection plans against the risks of contamination. Current bylaws require to delineate the total area from which groundwater is captured (the catchment), and a few capture zones which define protection perimeters. For example, a capture zone of 200 days is the area from which groundwater is captured by the well within 200 days. Several delineation methods are available. They range from the simple arbitrary fixed-radius to complex numerical modelling. A few analytical solutions have been developed for capture zones, but always for idealized conditions (Bear and Jacobs 1965; Grubb 1993; Chapuis and Chesnaux 2006; Chapuis, 2011).

Protecting water supplies involves predicting the fate of pollutants in groundwater. This is a difficult and uncertain exercise using solute transport theory, which involves effective porosity n_e, and longitudinal dispersivity, α_L. Laboratory reduced-scale models have verified that the theory is realistic for homogeneous materials. In nature, however, most aquifers are heterogeneous, and have to be studied using more complex theories and numerical models.

Effective porosity, n_e, denotes the fraction of the total volume of a saturated porous material that is used for movement of water. It excludes unconnected and dead-end pores and thus, is smaller than total porosity, n. Also called kinematic porosity, it differs from “specific yield”, S_y, which depends upon length of specimen, duration and type of drainage test, final suction, etc.

Some papers still confuse n_e and S_y despite the warning by Bear (1972). This confusion may come with the belief that a soil has a single n value, thus ignoring the concept of compaction. Let us consider two uniform soils, coarse sand and non-plastic silt: they may be tested at the same $n = 0.40$ and yield $n_e \approx 0.39$ for two laboratory column tracer tests. However, some drainage test may yield $S_y = 32\%$ for coarse sand and less than 5% for non plastic silt, which confirms that n_e and S_y are different physical parameters. Another type of confusion appeared in Riva et al. (2006) who proposed a linear correlation between ln (n_e) and ln (K), K being hydraulic conductivity. For the two soils, both may have $n = 0.40$ and $n_e \approx 0.39$, but their K values differ by a ratio of 10^3 to 10^4. Therefore, ln (n_e) and ln (K) cannot be linearly related.

Dispersivity is used to describe the dispersion of tracer mass. It has been studied in statistically correlated permeability fields, which have confirmed that dispersivity and heterogeneity are related. Gelhar and Axness (1984), Schwarze et al. (2001) and Dentz et al. (2011) proposed to correlate α_L and the variance, σ^2, of the K distribution when $\sigma^2 < 1$, K being the hydraulic conductivity.

A recent research with the Web of Science and three key words (aquifer tracer dispersivity) brought back 215 papers. Adding “effective porosity” gave only 22 of the 215 papers. This means that 90% of the 215 papers had no information on n_e, which is surprising. Another research with key words (aquifer tracer “effective porosity”) brought back 45 papers. A few had information for n_e, but most of these simply used n_e as an input for their numerical models. This confirms that n_e is not known a priori and has to be extracted from breakthrough curves (BTCs) by fitting data with 1D, 2D or 3D solutions with more or less parameters (Ptak et al. 2004). In laboratory tests (homogenized soils), n_e is lower than n but very close. In field tests, n_e is also lower than n, but by how much?

While α_L can be predicted by rough correlations (Chin 1986; Gelhar et al. 1992; Xu and Eckstein 1995, 1997) or scaling methods (Frippiat and Holeyman 2008), there is no reliable method to predict n_e. There are only curve fitting methods. This is unfortunate for consultants who simply assume some n_e value from their experience or the literature, and use it with some predicted α_L. Further, if a field tracer test is carried out, a theory may be chosen and used to extract n_e by curve fitting.

The lack of research on n_e is regrettable for those who have to protect drinking water wells. Also, most field breakthrough data are difficult to fit with theoretical models (Fernandez-Garcia et al. 2005; Pedretti and Fiori 2013). The theoretical study of tracer tests has advanced but it has become increasingly complex. Meanwhile, consultants have to guess the field n_e and α_L values (most studies) or estimate them by fitting the breakthrough data to some model (a few studies).

The goal of this paper is to reduce a gap between theoretical research and practical needs. It makes use of recently derived analytical equations (Chapuis 2015) for the hydraulically equivalent homogeneous aquifer (HEHA), thus for $n_{e\ HEHA}$ and $\alpha_{L\ HEHA}$ at field scale. The background is briefly presented, and then, the predictive equation for n_e is verified using experimental data.

Background

Chapuis (2015) assumed that different seepage velocities in stratified aquifers create dispersion, which results in large-scale values, $n_{e\ HEHA}$ and $\alpha_{L\ HEHA}$. Two problems were examined and solved. The first problem is rectilinear seepage, at constant hydraulic gradient i, in a stratified horizontal confined aquifer of constant thickness, where K varies only vertically. The second problem is for a well pumping the same aquifer at a constant flow rate Q, for radial steady-state seepage. The perfect well is vertical and fully penetrating. The radial groundwater flow converges towards the well.

Initially the non-reactive tracer concentration C is zero everywhere. Starting at time $t = 0$, the tracer enters the external boundary at a concentration C_0 (step function), which is maintained either forever or for a limited time. It is assumed that small-scale diffusion does not play a key role in the flow and transport equations. Pure convection is considered: the C_0 step function produces a piston flow in each layer. The resulting analytical equations for large-scale $n_{e\ HEHA}$ and $\alpha_{L\ HEHA}$ are then derived. Variations at the individual pore scale are not taken into account.

Chapuis (2015) solved the two simple problems first for a finite number of layers, and for the HEHA having the same flowrate for the same boundary conditions, thus a single K value equal to the averaged K_{ave} value. This is a frequent assumption, but the assumed homogeneity with K_{ave} is correct only for the flowrate. Then, Chapuis (2015) solved the two problems for a large number of layers. Each layer No. j had K_j and n_{ej} values. The K_j values followed a lognormal distribution of mean $\mu_{\ln K}$ and variance $\sigma^2_{\ln K}$ or they follow a normal distribution of mean μ_K and variance σ^2_K.

Moreover, the layers were assumed to have parallel grain size distributions, with the same stress and strain history, which yields local $nj = n$, and $n_{ej} = n_e$. The theory (Chapuis 2015) makes no assumption on spatial correlation. The water seeps parallel to stratification. The velocity field depends upon the $K(z)$ field, the constant gradient i (at all x values for the 1st problem, at any constant r value for the 2nd problem), and $n_{ej} = n_e$. The most conductive layer is the first to supply tracer mass, and the gradual input of all layers produces a breakthrough curve (BTC).

For a lognormal K distribution, Chapuis (2015) obtained a new solution that is close to the 1D solution for the advective-dispersive equation (Ogata and Banks 1961). For the HEHA, $n_{e\ HEHA}$ was obtained for $C/C_0 = 0.5$ at time $t_{50\ \ln K}$, which yielded

$$n_{e\,HEHA} = n_{ej}\frac{\exp(\mu_{\ln K})}{\exp(\mu_{\ln K} + (\sigma^2_{\ln K}/2))} = n_e\frac{\exp(\mu_{\ln K})}{K_{ave}} = n_e\frac{K_{50}}{K_{ave}} \qquad (1)$$

in which K_{50} is the value such as 50% of the K values are lower than K_{50}. Equation (1) confirms that $ne\ HEHA$ is smaller than the single n_e of each layer. It also confirms the frequently observed "early" tracer arrival in field tests. In general, $\mu_{\ln K}$ is between -11 and -7 (e.g., Chapuis 2013) whereas $\sigma_{\ln K}$ is between 0 (spheres having the same diameter) and about 2. As a result, Figure 1 shows how the ratio ($n_{e\ HEHA} / n_e$) varies in theory. Examples of tracer tests are given below to verify eq. (1).

In the case of a normal K distribution, the theoretical development gave (Chapuis 2015):

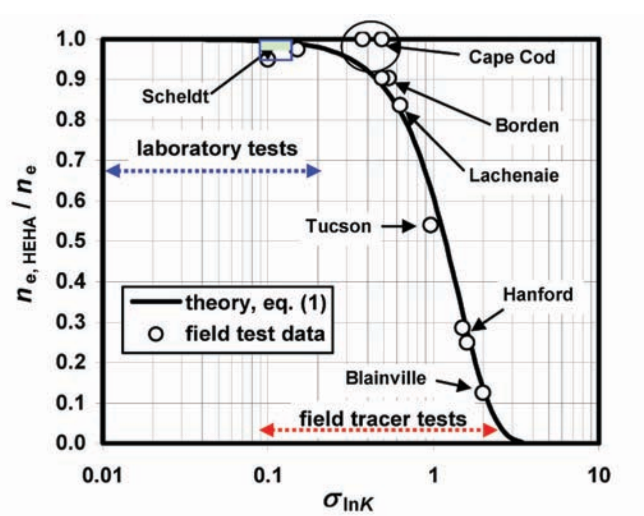

Figure 1. Variation of the ratio ($n_{e\ HEHA} / n_{ej}$) predicted by eq. (1) as a function of $\mu_{\ln K}$ (from about -11 to -7) and $\sigma_{\ln K}$ (from about 0 to 2).

This eq. 2 was also derived from the classical solution (Bear 1972). Thus, if the K distribution is normal, then

$$n_{e\,HEHA} = n_e. \tag{2}$$

$n_{e\,HEHA}$ is equal to the single n_e of each layer. This is commonly found with laboratory column tests using homogenized soils. However, field tests yield $n_{e\,HEHA}$ values that may be much lower than the n_e of each layer (Stephens et al. 1998). This contradicts the assumption of a normal K distribution. Therefore, experimental data support the assumed lognormal K distribution, which is usually observed with small- and middle-scale K values (Law 1944; Freeze 1975; Chapuis 2013).

Experimental verification

Laboratory tests

Consider a 1D laboratory tracer test. Homogenized sand was poured between two parallel walls. The sand was compacted as regularly as possible, for example in small layers about 2.5 cm high, using a free-falling tamper of defined mass and height of fall. At the end of the process, despite all the precautions, the sand layers yet have a small variation in K. The distribution of ln$K$ and its variance are estimated first, and then the resulting n_e is obtained with eq. (1).

The sand is defined by its grain size distribution curve (GSDC) and the roundness factor, RF, of its particles. The minimum and maximum values for n, n_{min} and n_{max}, or void ratio e, e_{min} and e_{max}, can be given by standard tests (ASTM D4253, D4254) or with the chart of Youd (1973). This chart was transformed into equations linking e_{min} and e_{max} to the sand coefficient of uniformity, C_U, and RF (Chapuis 2012a). Some variation in GSDC and compaction yields some variation in effective diameter d_{10} and void ratio e, which can be used to assess the K distribution. In the next example it is assumed that d_{10} = 0.16 ± 0.2 mm, and e = 0.52 ± 0.06.

Many methods were proposed to predict K. After assessing 45 methods, Chapuis (2012b) found that the most reliable for non-plastic soils are that of Hazen (1892) when coupled with Taylor (1948), Kozeny-Carman (Chapuis and Aubertin 2003), and Chapuis (2004). Here, eq. (3) is retained. It predicts K values between half and twice the experimental K values for the tests that avoided all of the common 14 mistakes of laboratory tests (Chapuis 2012b). Equation (3), where d_{10} is in mm, gives a K value of 2.17 x 10^{-4} m/s for the mean values of d_{10} and e:

$$K\,(cm/s) = 2.4622\left(\frac{d_{10}^2\, e^3}{1+e}\right)^{0.7825} \tag{3}$$

For simplification, a = 2.4622, b = 0.7825 and $d_{10} = x$; the logarithmic differential of eq. (3) is:

$$\frac{dK}{K} = -b\left[\frac{3de}{e} + \frac{2dx}{x} - \frac{d(1+e)}{1+e}\right] = -b\left[\frac{3de + 2e\,de}{e(1+e)} + \frac{2dx}{x}\right]. \tag{4}$$

Equation (4) is then used to assess the relative error (dK/K) resulting from the relative errors on d_{10} (dx/x) and e (de/e) when these values are small (≤ 10%), and also the relative uncertainty ($\Delta K/K$), where ΔK is the absolute value of dK. The equation for relative uncertainties is:

$$\frac{\Delta K}{K} = b\left[\frac{3\Delta e + 2e\,\Delta e}{e(1+e)} + \frac{2\Delta x}{x}\right]. \tag{5}$$

The numerical application for the previous sand data yields:

$$\frac{\Delta K}{K} = 0.44 \quad . \tag{6}$$

As a result, K = (2.2 ± 1) x 10^{-4} m/s. However, because the variation exceeds 20%, the direct calculation is preferred and gives: 1.36 x 10^{-4} ≤ K ≤ 3.27 x 10^{-4} m/s.

Similar developments can be made with the Hazen-Taylor and the Kozeny-Carman equations.

Corresponding normal and lognormal K distributions appear in Fig. 2. With μ_K = 2.17 x 10^{-4} m/s and σ_K = 4 x 10^{-5} m/s, eq. (1) gives α_L = 0.58 mm, and eq. (2) gives $n_e = n$. With $\mu_{\ln K}$ = –8.435 and $\sigma_{\ln K}$ = 0.180, eq. (1) gives $n_{e\,HEHA}$ = 0.984 n. Such a high ratio $n_{e\,HEHA}/n$ is regularly obtained with laboratory tracer tests for sand or clay (Sevee 2010), and also with field tracer tests limited to individual layers (Pickens and Grisak 1981). These results support the lognormal assumption.

Consider now a poorly prepared tracer test. The sand has large variations in d_{10} and e due to poor quality control. This may double the previous σ_K and $\sigma_{\ln K}$, and thus quadruple the variances, which gives α_L = 2.4 mm for the normal K distribution, α_L = 1 mm and $n_{e\,HEHA}$ = 0.937 n for the lognormal K distribution. Because the n value is inaccurate (poor control), the difference between n and n_e is inaccurate. Therefore, one cannot differentiate a well-prepared from a poorly-prepared laboratory tracer test. In addition, the differences resulting from the two assumed K distributions, normal or lognormal, are too small to tell which distribution should be preferred for a laboratory tracer test.

Field tests

Many field test data were collected (e.g., Gelhar et al. 1992), but most data are incomplete and cannot be used to assess the predictive eq. (1). Only a few complete sets of data were found. To interpret the Cape Cod tracer test in sand (LeBlanc et al. 1991; Garabedian et al. 1991; Hess et al. 1992), it was assumed that $n = n_e$ = 0.39. The ln(K) distribution had a variance of 0.14 (laboratory tests) and 0.24 (flowmeter tests), yielding $\sigma_{\ln K}$ = 0.37 to 0.49. Then, eq. (1) predicts that the ratio of $n_{e\,HEHA}/n_e$ exceeds

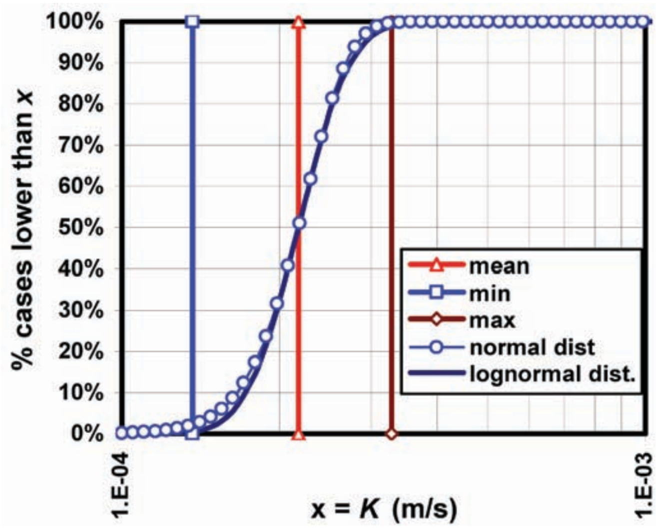

Figure 2. Example of a laboratory tracer test: the K range can be fitted with normal and lognormal K distributions, which are very close.

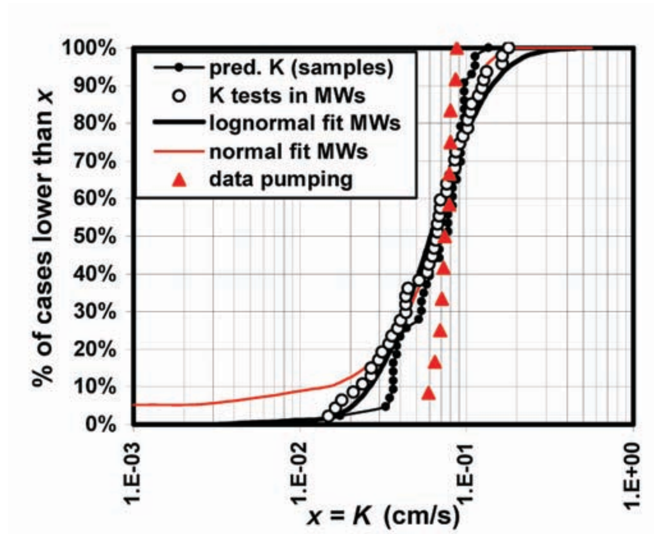

Figure 3. Experimental K distributions for the Lachenaie sand aquifer (Chapuis et al. 2005), with lognormal and normal best fits for the slug tests in monitoring wells after development.

Table 1. Lachenaie test: comparison of predicted and field values for large-scale K and n_{eHEHA}

		Lognormal K dist.		Normal K distribution		
Method	scale	K_{ave} (m/s)	n_{eHEHA}	K_{ave} (m/s)	n_{eHEHA}	type
Hazen-Taylor	small	7.4 x 10^{-4}	0.312	7.0 x 10^{-4}	0.40	predicted
Chapuis	small	7.5 x 10^{-4}	0.318	7.1 x 10^{-4}	0.40	predicted
Slug tests	middle	7.4 x 10^{-4}	0.324	6.5 x 10^{-4}	0.40	predicted
Pumping	large	7.4 x 10^{-4}	-------	7.4 x 10^{-4}	-------	experimental
Tracer	large	-----------	0.33	-----------	0.33	experimental

0.9, which in retrospect justifies the assumed equality.

The Borden test (Freyberg 1986; Sudicky 1986) was carried out in stratified sand of dry density ρ_d = 1.81 g/cm^3, and specific gravity of solids, G_s = 2.71, which give n = 33.2%. The test (Bales et al. 1997) gave $n_e \approx 30\%$, thus 90% of n (Mackay et al. 1986). The variance of ln(K) was either 0.29 (Sudicky 1986) or 0.24 (Woodbury and Sudicky 1991), which gave $\sigma_{\ln K}$ = 0.54 or 0.49.

The Tucson test gave $n_{e\,HEHA} \approx 0.5\,n$ (Stephens et al. 1998). The distribution K for the stratified aquifer was given in Zhang and Brusseau (1998). Ignoring the sub horizontal aquitard lenses, it was found for this paper that the aquifer sub layers ($K \geq 10^{-5}$ m/s) yielded $\sigma_{\ln K}$ = 0.96 (Chapuis 2016).

The Scheldt test was carried out in uniform sand (with a few silt-clay lenses) for which $n \approx 0.39$–0.40. The K values for the 14 layers forming the aquifer were given in Vandenbohede and Lebbe (2006). These were used for this paper to draw a distribution curve, which yielded $\sigma_{\ln K} \approx 0.10$–0.15.

The Hanford test was carried out in a sand-and-gravel aquifer (Bierschenk 1959) and gave n_e = 0.10. The sub layers had n values in the 0.35–0.40 range; the K values, between 3.5 x 10^{-5} and 3.5 x 10^{-2} m/s (Graham et al. 1981; Nevulis et al. 1989) were used for this paper to draw the distribution curve, which yielded $\sigma_{\ln K} \approx 1.5$–1.6.

Converging tracer tests were carried out in a sand aquifer at Lachenaie (Quebec) after steady-state seepage was reached for a pumping test. The tests yielded $n_{e\,HEHA}$ = 33%, whereas $n \approx 39$–40%. More detail, including the K distributions at small scale (samples) and middle scale (slug tests in monitoring wells) may be found in Gloaguen et al. (2001) and Chapuis et al. (2005). A lithium chloride solution was injected as a spike in the short screen of a monitoring well. Using the grain size distributions and the porosity, the small-scale K values were predicted using the methods of Hazen-Taylor (Hazen 1892; Taylor 1948; see Chapuis 2004) and that of Chapuis (2004). Each small scale K distribution was fitted with lognormal and normal functions, which correctly predicted the large-scale K and $n_{e\,HEHA}$ (Table 1). The middle-scale K values (slug tests) were also adjusted with lognormal and normal distributions (Fig. 3), which gave the predicted large-scale K_{ave} and $n_{e\,HEHA}$. All results (Table 1) show that the field K_{ave} and $n_{e\,HEHA}$ are better predicted by the lognormal K assumption than by the normal K assumption. However, the differences are small

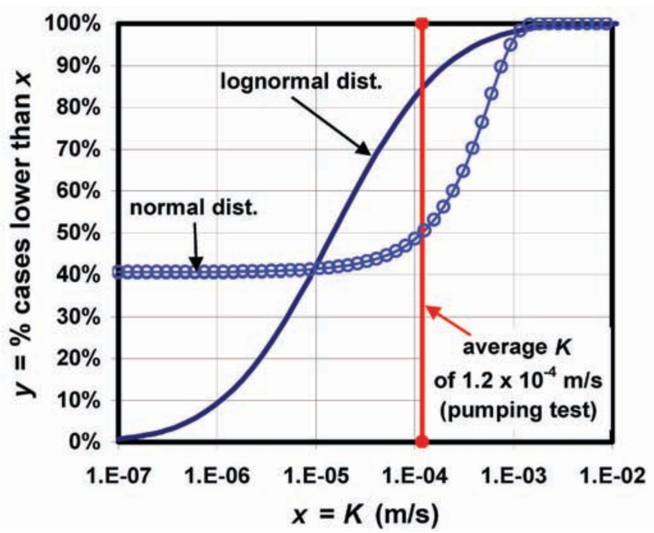

Figure 4. Blainville field tracer test (steady-state pumping): the normal and lognormal K distributions which fit K_{ave} are quite different. The lognormal K distribution is the only one that also fits the K range and the $n_{e\ HEHA}$ of 5% given by the tracer test.

because the variances are small for this fairly homogenous sand aquifer.

The Blainville test (Quebec) was carried out after reaching steady-state pumping conditions in an unconfined stratified aquifer (K_{ave} = 1.2 x 10^{-4} m/s), using a spike of lithium chloride in the screen of a monitoring well. The test gave $n_{e\ HEHA}$ = 5%, whereas $n \approx$ 40% for each layer. Trenches revealed 1- to 5-cm thick nearly horizontal layers, varying from pea gravel to silt, each soil being uniform, and K varying from 10^{-7} to 10^{-3} m/s. A lognormal K distribution, with $\mu_{\ln K}$ = -11.107 and $\sigma_{\ln K}$ = 2.039 (Fig. 4), covers the K range: it yields K_{ave} = 1.2 x 10^{-4} m/s and $n_{e\ HEHA}$ = 5%. For comparison, a normal K distribution with the same K_{ave} strongly differs from the lognormal distribution (Fig. 4), and predicts $n_{e\ HEHA}$ = 0.40 as for individual layers, a value much higher than the field value of 5%.

The data of those field tracer tests, for which all needed information could be gathered, appear in Table 2 and are plotted in Fig. 1: eq. (1) predicts correctly the experimental $n_{e\ HEHA}$ values.

Discussion

Despite academic progress, there is no reliable method to predict n_e. The missing information about n_e is regrettable for all specialists who need to predict the fate of contaminants and protect drinking water supplies. The objective of this paper was to verify a new analytical solution for the field n_e in stratified aquifers having a lognormal K distribution, under plane flow (Chapuis 2015).

Table 2 – Collected data for *n* and $n_{e\ HEHA}$, to assess eq. (1), as shown in Fig. 1.

No.	site	n	$n_{e,\ HEHA}$	$\sigma_{\ln K}$	$n_{e,\ HEHA}/n_e$
1	CapeCod	0.39	0.39	0.37-0.49	confused
2	Tucson	0.315	0.17	0.96	0.540
3A	Scheldt	0.39	0.38	0.15	0.974
3B	Scheldt	0.40	0.38	0.20	0.950
4A	Hanford	0.35	0.10	1.5	0.286
4B	Hanford	0.40	0.10	1.6	0.250
5A	Borden	0.332	0.30	0.54	0.904
5B	Borden	0.332	0.30	0.70	0.904
6	Lachenaie	0.40	0.33	0.66	0.825
7	Blainville	0.40	0.05	2.0	0.125

The approach was to use stratified aquifers with no local dispersion, in order to highlight the role of heterogeneity in velocity fields. This approach may seem outdated for those currently involved in research on tracer tests. In fact, the equations of this paper could have been developed in the 1970s or 1980s, but they were not before 2015. Recent theories on tracer tests have increased complexity, number of parameters, and yet they cannot predict effective porosity n_e and have limited predictive capacity for longitudinal dispersivity α_L. The existing 1D, 2D or 3D methods are only fitting methods, with many fitting parameters, which obscure physics and are too vague for practitioners.

As a result, this paper is the first one with a predictive equation for the effective porosity of the hydraulically equivalent homogenous aquifer (HEHA). This new equation is supported by field data.

The recently proposed analytical equations of Chapuis (2015) had shown that a lognormal K distribution can fully explain: (i) the early arrival of the tracer in field tests, using an equation providing the $n_{e\ HEHA}$ value; (ii) the increase of α_L with distance and also with the variance of the lognormal K distribution; and (iii) the long thick tail of field breakthrough curves. This paper has added an experimental verification of the predictive equation for n_e.

Conclusion

Current bylaws require to delineate the total area from which groundwater is captured (the catchment), and a few capture zones which are used to define protection perimeters. However, only a few national bylaws require field tracer tests converging towards the pumping well. As a result, groundwater professionals need to predict the values of effective porosity, n_e, and longitudinal dispersivity, α_L. However, despite academic progress, there is still no reliable method to predict n_e.

In this paper, recent analytical solutions for stratified aquifers have been used. These solutions were verified numerically using finite elements (Chapuis 2015). The predictive eq. (1) for $n_{e\ HEHA}$ involves the mean and variance of the ln(K) distribution: here, eq. (1) was verified experimentally. For laboratory tests with homogenized soils, the variability in K was shown to be low. The theory predicts that the large scale $n_{e\ HEHA}$ is almost equal to n, as usually observed in these laboratory tests. For field tracer tests, the spatial variation in K controls dispersion. The most permeable layers have a strong influence upon transport and dispersion and $n_{e\ HEHA}$ may be much lower than n. It is only for a low variance ($\sigma^2_{\ln K} \leq 1$) that normal and lognormal K distributions predict close breakthrough curves. Using the data experiments obtained by several authors, an excellent agreement is found between predicted and calculated (curve fitting) $n_{e\ HEHA}$ values.

Field tracer tests depend upon aquifer heterogeneity but also complex geometric conditions, time-variable boundary conditions, and processes such as sorption. However, the new findings here, for simplified cases, help to understand early tracer arrivals, and they are supported by field data.

Acknowledgments

The paper is the result of a research program subsidized by the Natural Sciences and Engineering Research Council of Canada (NSERC) to improve the quality of groundwater parameters.

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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

Sustainable field training in geology and geological engineering: Tunnelling and underground works field course in Greece

Nicholas Vlachopoulos and Efrosyni-Maria Skordaki

As with any graduate program within the realm of Geological and Geotechnical Engineering, there is a unique requirement to conduct field exercises in order to expose the students to the geology, real-life projects, experiences and working conditions with a view to reinforcing concepts that have been introduced in the traditional university classroom environment. Not only do the students benefit from such hands-on experiences, but the construction companies and contractors also benefit by positively influencing the students through the showcasing of their profession.

To this end, an international field course involving 4 universities was conducted from Monday, December 7th, 2015 until Sunday, December 13th, 2015. This graduate course is run annually by Dr. Paul Marinos (past-President, International Association of Engineering Geologists (IAEG)) at the National Technical University of Athens (NTUA). This year, as in previous years, the course was planned, organized and conducted in collaboration with the Civil Engineering Department at the Royal Military College of Canada (RMC), the Geological Sciences and Geological Engineering Department at Queen's University, NTUA and the Aristotle University of Thessaloniki (AUTH). An active, Canada-Greece inter-university collaboration in this regard has been established between these universities spanning more than 12 years. Organizers and instructors for the Canadian Universities were Dr. Nicholas Vlachopoulos, RMC/Queen's and Dr. Mark Diederichs, Queen's University. Eight graduate students from RMC-Queen's Canada participated in the course. This 'Canadian Contingent' was accompanied by eight graduate students from the graduate program of the Geology Department at AUTH, and twenty-one graduate students from the graduate program of the Schools of Mining and Metallurgical Engineering and Civil Engineering from the NTUA. Dr. Vassilios Marinos, Assistant Professor at AUTH also contributed with his expertise during the course.

Staff and Students from RMC, Queen's University, NTUA and AUTH participating in the field course.

The course involved circumnavigating Greece and visiting tunnelling sites (primarily road, rail, and mines) throughout the country. Greece is a country born of intense tectonic processes; being at the boundary of the African and Eurasian tectonic plates. Highly deformed and altered sediments and low grade metamorphic rock masses dominate the near surface environment creating a variety of technical challenges for tunnelling and slope stability related to modern infrastructure. The students certainly witnessed these issues first-hand. The underground construction works were conducted in limestones, clays, gneiss, molassic rocks, flysch, phyllites, ophiolites, basement schists and fault zones. The tunnels were at various stages of construction and the graduate student work along the way included geological model construction, seismic hazard prediction, ground classification and tunnel design with student presentations in the evenings among other deliverables.

There are certainly many active or recently completed tunnelling sites in Greece at the moment. Sites included on this course included: Kakia Skala Road & Rail Tunnels, Corinth Canal, Panagopoula Road Tunnel of Corinth-Patras Highway. Klokova Tunnel of the Ionian Highway, Gkiona Hydraulic Tunnel of Mornos-Athens, Kallidromo Rail Tunnel, Platamonas Tempi Road Tunnel, Thessaloniki Subway Tunnel, Euclid Station and Kalamaria Exten-

Crystalopigi Tunnel: conventional tunnelling students viewing the tunnel face minutes after explosives were detonated.

sion and Tunnel Boring Machines, Hellas Gold Mine sites in Chalkidiki, Skoureies and Olympiada, Tunnels of Egnatia Odos; multiple tunnels (20+), Sigma3, Anilio, Metsovo, Anthochori tunnels as well as the 'Great Cut' embankment, Crystallopigi Tunnels, multiple outcrops and rockmass characterization, Achellos diversion project by DEH (dams and power houses, tunnels) and Meteora Conglomerate formations.

Throughout the course, the graduate students were able to see the various challenges when tunnelling through materials with varying strengths and properties and in regions with inherent landslide as well as seismic risks. Of note, was the fact that the students had the unique chance to visit the Olympia and Skouries mine sites in Chalkidiki (another Canadian Connection as Eldorado Gold Corporation is a Canadian-based, intermediate gold mining company). The mines are rich in copper, gold, silver, and zinc in both surface and subsurface deposits. Greece has not played a significant role in the mining of such commodities for decades and as such, there are many unique challenges associated with the development of this site on a technical, social, and environmental level. It was an excellent opportunity for the student to witness how design and construction practices in mining differ from conventional highway or rail tunnels as well as the significant focus on worker safety and the environment.

Klokova Tunnel (Ionian Odos): conventional tunnelling – heading and bench - course visit.

A major contributor to the feasibility and success of such field courses is the buy-in and significant financial support provided by the tunnelling companies and contractors. Without such support and access to the underground works, these sorts of ventures would not be practicable. The companies see the need to help educate and expose the next generation of geological engineers or geoscientists to such sites. The direct access by the students to site engineers, workers and employees at all levels adds much value to the overall experience and compliment fully the objectives of the course. My experience in Canada has been that access to underground works of this nature are quite limited due (primarily) to liability considerations. I would welcome the opportunity to conduct such field courses in Canada in cooperation with tunnelling companies that would involve multiple and a diverse array of sites.

Framework for sustainable field course design

The field course was reviewed with current higher education research in mind and complemented with blended learning components (i.e. on-site instruction combined with on-line communication and critique of retained information) in order to enhance the instructional environment and provide a record of the field course's activities and lessons' learned for future courses. What makes fieldwork so valuable to learning geoscience? Pyle (2009) identifies the main goals of field courses as a) synthesis and application of knowledge; b) acquiring the field skills and techniques typically required for an entry-level, professional geologist; c) enculturation into the values and ethics of practicing geoscience; and d) exposing students to the variety of geologic phenomena they may encounter. Similarly, Mogk and Goodwin (2012) review arguments based on "practitioner's wisdom" claiming that field education yields improvements in students' knowledge and problem-solving skills, enhances students' ability to reflect on their own thinking (metacognition), generates positive feelings that lead to enhanced learning, offers direct and immersive experiences of geologic phenomena, and introduces students to professional practice" (Petcovic, Stokes & Caulkins, 2014).

Specifically, blended learning components of the field course involved: a) A pre-field exercise self-assessment, b) daily online journals posted on the course's website by the students tied to the information that was presented each day of the field exercise, c) critiques of online journals among peers, and d) a post-field exercise self-assessment (Table 1).

Much academic debate has been dedicated to determining the necessary balance of methods and tools to be included in a Geological Sciences and GeoEngineering program. Consideration of several factors is warranted, the most prevalent of these factors being the current state of industry and their requirements, technological advancements, sustainability education as well as instructional methods informed by ongoing education research. The re-design of the course to include the use of blended (onsite/online) learning as well as synchronous/asynchronous interactions was conducted with a view to enhancing the learning outcomes of the geological and geotechnical field exercises.

By utilising existing technology and pedagogy in field training, we sought to: a) Identify the diverse learning needs of the students and connect them to the learning outcomes of the Field Course, b) Investigate the value of the field exercise specifically for each student with a view to informing the design of future field exercises with a *learner-centered* approach, and c) Allow for the production of student-generated teaching material (discussion forum posts, student reflections, videos, photos) that would capture the field exercise activities through the eyes of the learners.

The results of the pilot study were in agreement with other researchers in the Sustainable Education realm in the sense that geological field training may also be in need of *"a redesigned educational paradigm that is in essence relational, engaged, ethically oriented, and locally and globally relevant."* (Stirling, 2001). By

Table 1. Blended Learning Component of Field Exercise (Description of Requirements).

Pre-Field Exercise Self-Assessment –Selected Questions	Online Journals:	Critical Reviews (Critiques) of posted Daily Reflections:	Post-Field Exercise Self-Assessment
How do you anticipate this field trip to support your learning in geoengineering? Include specific areas of learning that you would like to expand on (such as field work practices, identification of particular geological formations, field data collection etc.).	• Describe the following: -For each day of the technical tour, what were the most important technical characteristics of the sites that you visited? • Describe and critically reflect on the information that was presented onsite by course instructors, consulting engineers, technical company representatives etc. Was the information that was provided helpful to you? Why, or why not? What new did you learn? What specific areas/topics would you like to know more about?	• For each Critique, critically examine the content of a Daily Reflection post of your choice. • Are there any key technical characteristics about the information on the sites that are missing in the post you are evaluating? Are there any features about a particular site that you would like to highlight or elaborate upon? Feel free to share any new information such as a website link or bibliography that is relevant to the information presented in the post.	• Did the field exercise meet your learning needs as a geoengineering graduate student? • Did you gain new knowledge in the technical areas that you were hoping to expand on? Why or why not? • During this technical tour, did you visit any sites that attracted your research interest in particular? If yes, which ones? Why? Please explain.

combining: a) Specialized technical information provided by experts in the field, b) Assignments promoting student reflection on their actual learning during their participation in technical tours and onsite field exercises, and d) Synchronous/asynchronous peer collaboration and online critiquing and archiving of information, the authors set the framework for Sustainable Field Training in Geosciences/Geoengineering.

Not only was this field course a memorable one from a technical perspective, but it was also an experience that the students will remember as a cultural exchange. For the duration of the visit, the generosity as well as warm culture of the Greek people was a highlight, as at no moment was there a lack of hospitality or kindness. A common takeaway was about embracing culture of the Greeks who have a passion for learning, teaching, and expanding their knowledge base. This was highlighted by the fact that due to the enthusiasm of Greek GeoProfessionals (in particular, Hydro Greece (D.E.H.) staff that travelled over 200 km in order to provide students access to an underground power generation cavern). In this respect, there was no evidence of any crisis in Greece. The Canadian Contingent enjoyed learning about the Greek culture, and feasting in their delicious and varied cuisine. This field course helped all those that were privileged to take part in it to grow personally and take a little bit of Greece back to Canada with them; as Dr. Paul Marinos himself put it, "You are now all Greek!"

This type of international collaboration between these institutions that has spanned more than a decade has won high praise from the Embassy of Canada to the Hellenic Republic, specifically, from Ambassador Keith Morrill himself. We look forward to future venues and collaborations with a view to improving such experiences for our graduate students – in a sustainable fashion.

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Nicholas Vlachopoulos

Civil Engineering Department, Royal Military College of Canada, Kingston, Ontario K7K 7B4
T: 613-541-6000 x 6398
email: vlach@rmc.ca
Nicholas is a professor with the Royal Military College of Canada (RMCC) within the Civil Engineering Department and Director of the RMC Green Team.

Efrosyni-Maria Skordaki

Civil Engineering Department, Royal Military College of Canada, Kingston, Ontario K7K 7B4
T: 613-541-6000 x 8779
email: Efrosyni.Skordaki@rmc.ca
Efrosyni-Maria is geosciences research staff with the RMCC within the Civil Engineering Department and instructional designer at the Canadian Forces School of Communications and Electronics.

Case History IX
Part 2

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 following is the conclusion to Part 1 of Case History IX which appeared on pages 51 – 54 of the June issue of Geotechnical News

Following this conversation Mr. Brown put the following, in capital letters, as a note to the Foundation Plan drawings:

Site to be preloaded as per Smith's Laboratory Report dated June 18, 1979. Preload to remain in place 8 weeks or until settlement ceases.

Thus it was that part of the defendant's preliminary soils investigation report and some of Mr. Jones remarks on the telephone to Mr. Brown concerning settlement time became transformed into specific construction specifications on the final foundation plan. I am satisfied that the defendant's personnel never approved of this notation, and that they remained unaware of it until after construction had been completed and the settlement had occurred.

Mr. Brown was candid in conceding his responsibility for the project, and that there had been some oversight on the Contractor's part.

He said he knew that settlement gauges must be used in order to be sure when settlement has ceased, that the preload should not be removed until settlement has ceased, and that a preload must extend beyond the edges of the actual building envelope. He said he assumed from the defendant's report that a properly designed preload was then already in place. While the report was described as "preliminary" it did not seem preliminary in substance, he said, because it contained specific recommendations and conclusions. He thought the defendant would have known, as a result of his telephone conversation with Mr. Jones, that the report was being used for design purposes. He said he assumed from this conversation that monitoring the preload would not be necessary if it remained in place for eight weeks.

While maintaining that the Contractor was not qualified to design a preload, Mr. Brown agreed that it had the responsibility to see that a proper foundation design was done, including the preload, and to inspect the site. No preload design was in fact provided to the plaintiff, nor did the Contractor or any engineer on its behalf, inspect the site before building commenced.

Mr. Doe testified that he received the plans from the contractor on June 27 and read the note reproduced above. He understood it to mean that the individual truckload piles of sand dumped within the building envelope constituted a proper preload, and that all he had to do was leave them there for eight weeks and the ground would be ready to support the floor.

Mr. Doe said he assumed that the Contractor had drawn correct conclusions from the defendant's report. For that reason, he said, he put reliance on the report, and did what he thought it said.

Should the defendant then, in preparing its report, have foreseen the possibility that this might happen?

Mr. Smith believed, quite correctly, that there was an engineer in overall charge of the project. His firm was retained in a very limited way to do a basic soils test for $400. He identified the investigation as preliminary only. He assumed that the site was to be inspected by the supervising engineer, and that the engineer would have some knowledge about preloading. The report said there were "stockpiles of river sand deposited on the building site", and that this sand "constituted suitable preloading material". Its only recommendation on preloading was "use one foot of this sand as surcharge for each 95 psf of dead and live load anticipated on this floor". It did not say there was a preload in place, nor did it say how to create or employ one. The report said only that there was suitable material on site and how much would be needed.

I have concluded that the defendant could not have been expected to foresee the possibility that an engineer in charge would refer to this report as an instruction on preloading, or that he would interpret it to mean that a properly-designed preload was in place, and not make any inspection. The report says nothing about the shape or position of the preload, or how to know when to remove it. Nor is there mention of the volume which must be removed, as opposed to that to be left in place as fill.

Should remarks made by Mr. Brown, then, in his telephone conversation with Mr. Jones, have alerted the defendant to the possibility that the report, or Mr. Jones comments, might be used by Mr. Brown as they were?

Mr. Jones could not, I think, have guessed, without being told, that the report was being relied on as indicating that a properly-designed preload was already in place. Nor do I think he could reasonably be expected to foresee that the Contractor intended to put on the plan a note capable of being interpreted as meaning that monitoring of settlement was unnecessary—that merely leaving a preload in place for eight weeks would be sufficient to assure that the necessary settlement had occurred. I say that particularly because I accept that Mr. Jones did mention the need to monitor in his conversation.

I cannot therefore find that there was negligence on the part of the defendant up to this point, which, had the defendant's involvement then ceased, could be said to have contributed to the ultimate failure.

The pile-driving inspection

During July Mr. Doe retained a pile-driving company to put in perimeter piles in accordance with the contractor's foundation plan, and asked the defendant to send someone to supervise the operation.

Mr. Jones attended at the site for this purpose July 12 and 13. He found the sand on the site had been arranged so as to make room for the pile driving crew to do that work around the perimeter. Mr. Doe spoke to Mr. Jones about the preload during the course of the pile-driving, and he drew Mr. Jones' attention to the failure of the building next door. He testified in court that he asked Mr. Jones how the preload seemed and that Mr. Jones replied that it was "fine" and to leave it on for eight weeks. In cross-examination Mr. Doe said that this answer was given in a "off-hand" way, but that he relied on it. He said he relied also on the defendant's report in coming to the conclusion that preloading was being properly done. He denied that Mr. Jones mentioned the use of settlement gauges during this brief exchange.

On examination for discovery, Mr. Doe had said he relied solely on what he was told by Mr. Jones on this occasion, so far as the preloading was concerned, and not on anything contained in the report. Mr. Doe also said on discovery he had understood, until he spoke to Mr. Jones on this occasion, that he would have to keep the preload in place for longer than eight weeks if settlement had not ceased when the eight weeks was up.

Mr.Jones' evidence was that Mr. Doe pointed at this meeting to the sand and asked what Mr. Jones thought of the preload and that he answered that it seemed high enough. He said he asked Mr. Doe how long it had been on and Mr. Doe indicated about three months and asked if that was long enough. He said he replied that it might be but that one would have to use settlement gauges to be sure. He said Mr. Doe asked if that was really necessary and Mr. Jones replied that Mr. Smith always used them.

I accept Mr. Jones' evidence as a reasonably accurate account of the exchange which took place between them that day.

I cannot find that this casual conversation should have caused Mr. Jones to realize he was being relied on to warn the plaintiff of any inadequacy there might be in the preloading procedure. His firm had not, of course, been retained for that purpose and his visit had nothing to do with it. The approval which he expressed in an off-hand way could only have related to the quantity of sand. It could not have related to the configuration—which had in any event been disturbed for the pile-driving- nor to whether settlement had ceased. I accept Mr. Jones' evidence that he told Mr. Doe that settlement must be monitored in order to know whether it had ceased.

Mr. Doe's prior understanding, as expressed on examination for discovery, that he would have to leave the preload in place if settlement was still taking place when the eight-week period mentioned on the foundation drawings expired, was a manifestly reasonable one which could not reasonably have been displaced by this conversation with Mr. Jones.

I.cannot say that Mr. Jones was negligent in the remarks which he made to Mr. Doe on this occasion.

Despite his understanding that settlement had to be checked, Mr. Doe proceeded to level out the preload without knowing whether it was still settling. He spread the sand over the actual building envelope and the adjacent parking area so as not only to fill in inundations caused by the preloading but to raise the building envelope to a higher elevation. He then had the pile-supported cement perimeter foundations built and thereafter handed the job over to the contractor for construction of the building.

The only further involvement of the defendant during the foundation phase of the project was the conduct of laboratory tests on concrete and pile-cap samples provided to it. This did not involve work at the site.

Ought the defendant of its own volition to have volunteered a warning about the preload during this period?

The plaintiff says the defendant ought to have realized that Mr. Doe was inexperienced and that he might be proceeding on a dangerous course—that the preload probably had not been properly shaped, and was not being monitored—and should have given him a warning. The fact that the defendant had not been engaged for preload design or supervision is no answer the plaintiff says, to this allegation of negligence in failing to give some sort of warning during or after Mr. Jones' July 12-13 visit.

When he returned from the pile-driving operation, Mr. Jones told Mr. Smith of his conversation with Mr. Doe and said that he saw no settlement gauges. There can be no doubt that Mr. Smith, had he been asked to give his advice in the matter at this stage, ought to have expressed doubts on whether the preloading had been competently done. He had himself been at the site briefly during each of Mr. Jones' visits. On neither occasion was there a properly-shaped or properly-positioned preload, although this could on both occasions to some extent be explained. He had no knowledge that there was a proper preload in place, and good grounds for doubting it. He knew from Mr. Jones that it was unlikely settlement was being monitored.

Mr. Smith testified that as a consultant with a strictly limited engagement he had no justification for involving himself.

He had, of course, been retained for restricted purposes. He knew there was a supervising engineer in charge and had confidence in that engineer's ability. A supervising engineer is taken to accept responsibility for all necessary engineering functions which have not been delegated to others. Had he been in Mr. Brown's shoes, Mr. Smith said, he would not have appreciated gratuitous interference from a soils consultant in a matter in which the consultant had not been engaged. Mr. Smith felt that the Contractor had chosen either to use its own resources or take advice elsewhere with respect to the preloading, and it did not seem to him that he could properly involve himself in the matter.

I have no doubt that there are circumstances in which a professional man may have a duty to warn in connection with matters about which he has not specifically been engaged. But where he knows that another member of his calling has been retained in a matter it is difficult to conceive of such circumstances—short, in any event, of those involving hazard to life—in which he would be under a duty to involve himself without first receiving a formal request for his opinion. The casual enquiry made of Mr. Jones by Mr. Doe seems to me to have fallen far short of what an engineer could regard as such a request.

I cannot therefore find that the defendant was at this stage, under a duty to make gratuitous enquiries, to offer gratuitous advice, or to warn the plaintiff of any risk to which it might be exposed.

The inspector's warning

The last on-site investigation, conducted by the defendant, occurred during September, was directly related to the possibility of settlement, and resulted from an expression of concern by the municipal inspector that the plaintiffs building might suffer the same fate as that next door.

The municipal building inspector suggested to Mr. Brown, the: contractor's chief engineer, that he should satisfy himself that his design would not result in the sort of settlement which had occurred in the building on the adjoining property. At this point the shell of the plaintiffs building was largely completed, with the roof in place, but the concrete floor had not yet been poured. Mr. Brown telephoned Mr. Smith to pass on the message. He asked Mr. Smith if he would visit the site and see if there was any reason for such concern. Mr. Smith agreed and said he would call Mr. Brown back if he concluded there was.

Mr. Smith looked at the two buildings and took some photos. He concluded there was nothing to suggest that the plaintiffs building might be in any danger. He decided there was no reason to call Mr. Brown.

The reason Mr. Smith concluded that there was no need for concern was because he saw no visible evidence of distress in the case of the plaintiffs building. In the case of the building next door, on the other hand, there were obvious signs of settlement. There was a gap, clearly visible from the outside, between the ground and the pile-supported concrete perimeter wall foundations. There was also clear evidence of settlement of the floor inside. No settlement was evident at the perimeter of the plaintiffs building, and it had as yet no floor. Finding no similarity in the condition of the two buildings, Mr. Smith concluded that there was no need for concern.

Mr. Smith seems to have viewed his task on this occasion as that of an observer. He does not seem to have felt that it was his duty to make enquiries. I must ask whether he was justified in adopting this view.

It seems to me unlikely there could be evidence of settlement at that point on the plaintiffs site, even if the foundation conditions there were as defective as those next door. Only a month had passed since the sand had been spread out and the pile-supported perimeter foundations built. Since the floor slab was not yet poured, no weight had yet been imposed on the newly-created grade. The next-door building, on the other hand, had been completed and in use for more than a year. The preloaded soils there had long been under sustained stress, while the plaintiff's foundations had yet to be tested.

I have concluded that a visual inspection could do little, in these circumstances, to answer the question which the inspector had posed and which Mr. Brown had passed on to Mr. Smith.

In the light of what he knew and did not know about the preloading, and of what he ought as a consequence to have questioned, I think Mr. Smith had a duty to make enquiries before he could justifiably say that the plaintiffs building would not suffer the same fate as its neighbour. I think he had to know what sort of preloading was done in each case; certainly he had to know what sort of preloading had been done on the plaintiffs site. If he did not wish to pursue the matter beyond a visual inspection I think he was bound to tell the contractor that he could not answer the question put to him.

By his silence Mr. Smith implied that there was no need for concern. In an engineer having that special expertise and with the knowledge which he did have, to be silent in the circumstances seems to me to fall short of the appropriate standard of care.

I find there was negligent conduct also on the part of the contractor in failing to communicate to Mr. Smith information which it had and which it ought to have realized Mr. Smith would need in order to answer the question it asked. Mr. Brown should have disclosed the preloading instructions appearing on the foundation plan; he should also have said that the contractor had not inspected the preload and did not know how it had been done.

While there seems to me clearly to have been negligence on the part of the plaintiff in the conduct of the preloading, I cannot say it contributed to the defendant's failure to discover and warn of the danger following the building inspector's enquiry. That, I conclude, was due in equal parts to the negligence of the contractor and the defendant. But for their negligence, I find that the plaintiff would have been warned of the grave danger in proceeding with the floor slab, and would have taken remedial action.

Corrective measures which would have been instituted at that stage would necessarily have been less costly than those which had in the end to be undertaken after settlement occurred.

Conclusion

The only negligence of the defendant which I find to have been proved is in its misrepresentation by silence following the specific engagement by the contractor in September to advise on risk of settlement.

I have found the defendant was responsible as a consequence for 50% of the damages suffered because of the delay in remedial action between September, and the time when settlement took place. I have reached that conclusion because I find: (a) that at no time prior to September, 1979, did the defendant have reason to believe it was being relied on for professional advice as to the design, application, monitoring or removal of the preload; (b) that at no time prior to September, 1979, did the defendant give any advice on preloading which, properly considered, could have misled the person for whom it was intended; and (c) that no duty to warn rested on the defendant prior to September, 1979, because until that point it had not been engaged to give preloading advice and knew that another engineer was in charge.

I cannot say that use of the "mixed'' foundation design was in itself contrary to competent engineering practice, even though it is plain that some engineers would have recommended against it. The evidence suggests that the system is one which, with competent design and application, could on this project have achieved a satisfactory result.

The reason the floor failed in this case was that the preload had been improperly shaped, irregular in height, not properly positioned over the building envelope and only partly removed, and perhaps also because settlement had not been satisfactorily completed.

It seems to me that the plaintiff and the contractor may have wished to avoid incurring the cost of obtaining preloading advice formally from a soils engineer. The enquiries made of the defendant by the contractor and Mr. Doe in June and July seem to have been carefully calculated not to assign responsibility to the defendant for the preload. Those enquiries may well have been cast in an informal way in order to avoid such a commitment as would justify a charge. I do not think a professional man can be made responsible for the work of others by carefully limited enquiry, or mere casual reference. Nor, I think, can it be expected that he will always hedge gratuitous responses to such informal enquiries by disclaiming responsibility.

The parties have agreed that the court should deal with apportionment of liability under the Negligence Act, R.S.B.C. 1979 Chapter 298, whether it be in contract or tort. It seems to follow that I need not be concerned whether the duty which I have found to be breached arose out of the original contract between plaintiff and defendant or out of the general law of negligence, nor do I think I need consider whether the duty breached by the defendant was one in respect of which the contractor might have claimed against it for indemnity or contribution, rather than a duty owed directly to the plaintiff.

Judgement

The total damages claimed are $98,667, consisting of $65,708 as cost of repair, and $8,251 as lost tenants' contributions to municipal taxes and $24,708 as lost rental income during the period for which the premises were incapable of occupation because of settlement and repair work.

I have endeavoured to segregate the costs and losses which would probably have been saved or prevented had the problem been identified before the floor had been poured and the interior of the building completed. I approximate the avoidable repair costs at $34,000, and I would attribute one-half of the 14-month period of rental loss to this delay in identifying the problem- in representing about $12,000 in lost income—and also one half of the resulting additional municipal tax burden, that is to say about $4,000. The delay in identifying the problem which was to result in failure accounts for roughly half of the total damages attributed to the failure. Thus I have said that the defendant is equally responsible with the Contractor for that additional cost and loss, or for about 25 percent of the total damages claimed.

I would have allocated the total fault for the failure of the building and resulting damages 25 percent against the defendant, 25 percent against the plaintiff and 50 percent against the Contractor and would have allocated the costs of the action in the same proportions. I find the plaintiff to be entitled as against the defendant to 25 percent of the total damages, or $24,667, plus 25 percent of the plaintiffs taxable loss, less 75 percent of the defendant's taxable costs. The plaintiff is entitled in addition to pre-judgement interest, at the rates awarded from time to time during the relevant period by the District Registrar on default judgements. The relevant period may, I think, fairly be set as being from March 1, 1980, to today's date as to one-half of the damages, and from September 1, 1980 to today's date as to the remainder.

Since the allocation of the damaged items which I have made was not addressed in argument, I would be glad to hear counsel if either party should feel that I have erred in apportioning the loss and expense incurred as between that which would and would not have been experienced had remedial measures been taken earlier, or in any other matter of calculation.

June 6, 1983.

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KEY DATES

JULY 22, 2016
Deadline for full paper submissions

JULY 31, 2016
End of early bird registrations

OCTOBER 2, 2016
Ice Breaker reception

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