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GEOTECHNICAL*news*

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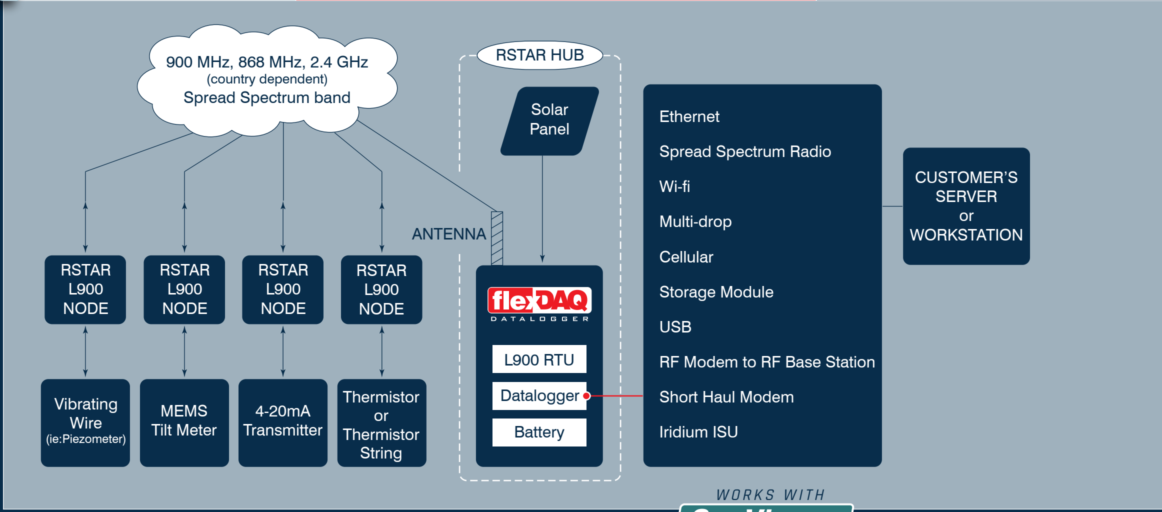
DIAGRAM

Typical RSTAR configuration for the RSTAR L900 System.



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- Ultra-low quiescent power. RSTAR Nodes powered by 1 lithium 'D' cell (up to 7 years of life).
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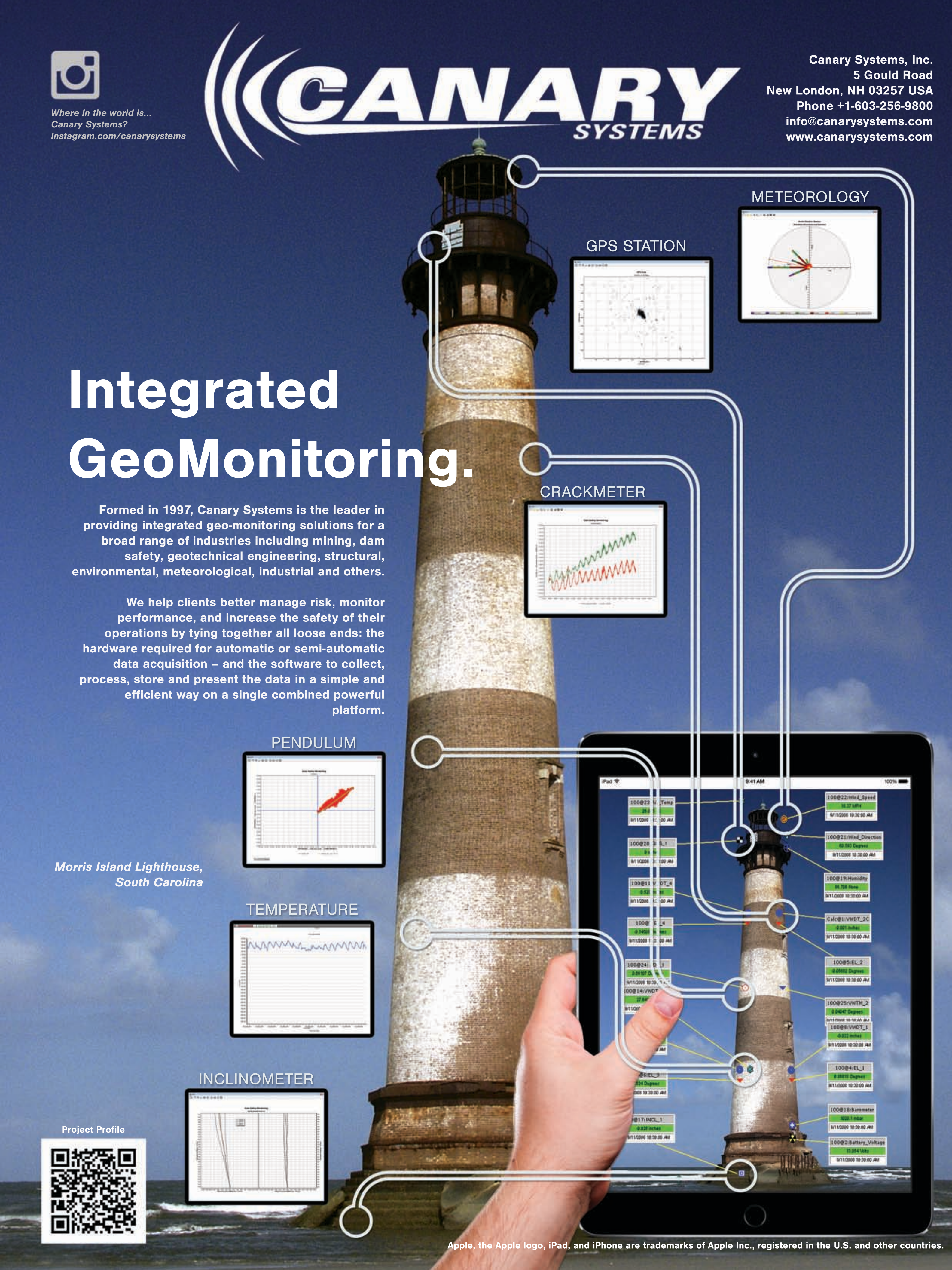
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CONTENTS

Message from the President

Doug VanDine, President of Canadian Geotechnical Society

It seems odd to me, as the President of the Canadian Geotechnical Society, that there is no commonly accepted definition of "geotechnical" in Canada. As far as I am aware, only one organization has tried to define the word. In 2013, the Association of Professional Engineers and Geoscientists of British Columbia's Task Force on Geotechnical Engineering defined "**geotechnical engineering**" as "**the application of principles of soil mechanics and/or rock mechanics, and related applied geological sciences**". I believe a similar definition could also apply to the broader "geotechnical profession". If anyone knows of another, commonly accepted Canadian definition of "geotechnical", I would be pleased to hear from you (*president@cgs.ca*).

Why is there no commonly accepted definition of "geotechnical" in Canada? I believe one reason is because geotechnical professionals come from and have a wide variety of backgrounds such as civil engineering, geological engineering, mining engineering, geology and/or physical geography. As such, geotechnical professionals aren't registered in their respective provinces or territories as "geotechnical engineers" or "geotechnical geoscientists".

A second reason might be that the geotechnical profession is extremely broad. Typical geotechnical activities include surface and subsurface site investigations; insitu and laboratory testing; development and analysis of models of near-surface and subsurface conditions; engineering design; construction, inspection and monitoring; operation and maintenance; research and development; and management.

Typical geotechnical project types include foundations related to onshore, near-shore and off shore structures; retaining walls; dams, reservoirs and dam safety; embankments and earthworks; slope stability and landslide hazard and risk assessments of natural and engineered slopes; ground improvement; dewatering; transportation and energy infrastructure; tunneling and underground works; pipelines and buried cables; resource development including minerals, oil and gas and groundwater; seismic response and liquefaction; materials testing; use of geosynthetics; geoenvironmental applications; and forensic investigations.

Therefore, because of the extremely broad nature of the profession, it's very difficult to define what we do and who we are. Why should there be a definition of "geotechnical" in Canada? I will leave that for the next President's Message, but suffice to say, I think it's important.

On other matters, I am pleased to report that CGS activities are moving right along. **Dr Nick Sitar**, from the University of California – Berkeley, recently completed his marathon Cross Canada Lecture Tour. The fall 2015 Cross Canada Lecturer will be announced shortly. The deadline for CGS Awards nominations has just passed, but the deadline for nominating CGS members for EIC Awards, Honours and Fellowships is still ahead of us, closing on **July 1**. More information on the EIC nominations can be found elsewhere in this issue of Geotechnical News.

Jean Côté and his local organizing committee for the **68th CGS Annual Conference and the 7th Canadian Permafrost Conference (GéoQuébec 2015)** *http://www.geoquebec2015.ca/EN/* report that they received a record number of abstracts for this year's conference. Papers have been submitted and are presently under review. For those planning to attend, registration with early-bird prices is now open. I know this is going to be an excellent conference, and I hope to see you all in beautiful Quebec City in September.

For those who really like to plan ahead, the 69th conference will be held in Vancouver in 2016 and the 70th conference will be held in Ottawa in 2017. More about those conferences later.

I have now been your President for just less than six months, and I can tell you that I am very impressed by the depth and breadth of this fine society and the excellent support that is provided by CGS Headquarters (**Michel Aubertin, Wayne Gibson and Lisa McJunkin**) and the myriad of dedicated volunteers that keep it going. Thanks to all.

Until next time.

Provided by Doug VanDine
President - 2015/2016

Message du président

À titre de président de la Société canadienne de géotechnique, il me paraît étrange qu'il n'y ait pas de définition couramment acceptée pour le mot « géotechnique » au Canada. À ma connaissance, une seule organisation a tenté de définir ce mot. En 2013, un comité spécial (Task Force on Geotechnical Engineering) de l'Association of Professional Engineers and Geoscientists of British Columbia a défini la « **géotechnique** » comme « **l'application des principes de la mécanique des sols et/ou de la mécanique des roches et des sciences connexes liées à la géologie appliquée** ». Je crois qu'une définition semblable pourrait également s'appliquer à l'ensemble de la « profession géotechnique ». Si quiconque connaît une autre définition canadienne couramment acceptée de la « géotechnique », je serais heureux que vous m'en fassiez part (*president@cgs.ca*).

Pourquoi n'y a-t-il pas de définition couramment acceptée de la « géotechnique » au Canada? Je crois que cela s'explique entre autres par le fait que les professionnels qui oeuvrent en géotechnique viennent de divers horizons et ont des parcours très différents, comme le génie civil, le génie des mines, le genie géologique, la géologie et/ou la géographie physique. Les professionnels de la géotechnique ne sont donc pas inscrits dans leurs provinces ou territoires respectifs comme des « géotechniciens » ou des « géoscientifiques ».

Cela peut également être parce que la profession géotechnique est extrêmement vaste. Habituellement, les activités géotechniques comprennent la reconnaissance des sols de surface et en profondeur; les essais en laboratoire et en place; la conception et l'analyse de modèles pour les conditions de surface et en profondeur; la conception des ouvrages; la construction, l'inspection et la surveillance; l'opération et l'entretin; la recherche et le développement; et la gestion.

Les projets géotechniques usuels comprennent les fondations de structures sur la terre ferme, près des côtes et en mer; les murs de soutènement; les barrages, les réservoirs et leur sécurité; les remblais et les ouvrages en sols et enrochement; la stabilité des pentes, les risques de glissement de terrain et l'évaluation des risques pour les pentes naturelles et aménagées; l'amélioration des sols; le pompage et l'assèchement; les infrastructures énergétiques et de transport; les tunnels et les travaux souterrains; les pipelines et les câbles enfouis; l'exploitation des ressources, y compris les minéraux, le pétrole le gaz naturel, et l'eau souterraine; la réponse séismique et la liquéfaction; les essais sur les matériaux; l'utilisation des géosynthétiques; les applications géoenvironnementales et les investigations plur identifier les causes d'une défaillance.

Par conséquent, compte tenu de la nature extrêmement vaste de la profession, il est très difficile de définir ce que nous faisons et qui nous sommes. Pourquoi devrait-il y avoir une définition de « géotechnique » au Canada? Je répondrai à cette question dans mon prochain message du président; il suffit de dire maintenant que je crois que c'est important.

À propos d'autres sujets, je suis heureux de signaler que les activités de la SCG vont bon train. Le **Dr Nick Sitar**, de l'Université de la Californie, à Berkeley, a récemment terminé son marathon dans le cadre de la Tournée de conférences pancanadiennes. Le conférencier de la tournée de l'automne 2015 sera bientôt annoncé. La date limite des candidatures pour les prix de la SCG vient de passer, mais l'échéance pour les candidatures de membres de la SCG pour les prix, les distinctions et les bourses de recherche de l'ICI est à venir, soit le **1er juillet**. De plus amples renseignements sur les candidatures pour l'ICI se trouvent dans un autre article de ce numéro de Geotechnical News.

Jean Côté et son comité organisateur local pour la **68e conférence annuelle de la SCG et la 7e conférence canadienne sur le pergélisol (GéoQuébec 2015)** *http://www.geoquebec2015.ca/fr/* signalent qu'ils ont reçu un nombre record de résumés pour la conférence de cette année. Des articles ont également été soumis et sont actuellement examinés. Pour les personnes qui prévoient y assister, la préinscription est commencée. Je sais que ce sera une excellente conférence, et j'espère tous vous voir dans la magnifique ville de Québec en septembre.

Pour ceux qui aiment vraiment planifier leurs activités, la 69e conférence aura lieu à Vancouver en 2016 et la 70e, à Ottawa, en 2017. De plus amples renseignements sur ces conférences vous seront fournis ultérieurement.

Je suis maintenant votre président depuis un peu moins de six mois et je peux vous dire que je suis très impressionné par le sérieux et l'étendue de cette excellente société et par le remarquable soutien qui est offert par le siège social de la SCG (**Michel Aubertin, Wayne Gibson et Lisa McJunkin**) et la myriade de bénévoles dévoués qui en assurent le bon fonctionnement. Merci à tous.

À la prochaine!

From the Society

Call for Nominations for 2015 Awards and Fellowships Engineering Institute of Canada (EIC)

As a constituent Society of the **Engineering Institute of Canada** (EIC), CGS members are eligible for awards and fellowships of the EIC which are summarized below. CGS members are encouraged to submit EIC nominations of fellow members to CGS Headquarters by **July 15, 2015.**

Nominations must include:

1. a completed EIC Nomination Form which is available from http://eic-ici.ca/honours_awards/)
2. a nomination letter

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

Prix ou distinction	Courte description/Commentaires
Médaille Sir John Kennedy	Pour un service exceptionnel rendu à la profession ou pour des contributions dignes de mention au domaine de la science de l'ingénierie ou au profit de l'ICI. Plus prestigieux prix de l'ICI.
Médaille Julian Smith	En reconnaissance d'une contribution au développement du Canada.
Médaille John B. Stirling	Pour des qualités de chef et des services émérites rendus à l'ICI et/ou à ses sociétés membres, à l'échelle nationale.
Médaille CP Rail Engineering	Pour les qualités de chef et le service rendu dans les régions et les chapitres de membres des sociétés membres de l'ICI.
Médaille K.Y. Lo	Pour des contributions remarquables au domaine de l'ingénierie au niveau international, comme la promotion de l'expertise canadienne à l'étranger, la formation d'ingénieurs étrangers, un service exceptionnel rendu à des organisations d'ingénierie internationales et l'avancement d'une technologie d'ingénierie reconnu sur la scène internationale.
Bourse de recherche de l'ICI	Pour l'excellence en ingénierie et des services rendus à la profession et à la société.
Membre honoraire	Pour les non-membres de l'ICI et de ses sociétés membres, et occasionnellement pour des personnes qui ne sont pas des ingénieurs, qui se méritent cette remarquable distinction en raison de services rendus au domaine de l'ingénierie et à la profession de l'ingénierie au Canada.

3. the nominee's CV, and
4. supporting letters from colleagues, preferably Fellows of the EIC (FEIC).

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

The Canadian Geotechnical Society
8828 Pigott Road
Richmond, BC
V7A 2C4, Canada,
Tel: (604) 277 7527; 1 (800) 710-9867; Fax: (604) 277-7529
E-mail: *cgs@cgs.ca*

Appel de candidatures pour les prix et bourses de recherche 2015 Institut canadien d'ingénierie (ICI)

À titre de société membre de l'**Institut canadien des ingénieurs** (ICI), les membres de la SCG sont admissibles aux prix et médailles de l'ICI décrits ci-dessous. Les membres de la SCG sont encouragés à soumettre la candidature de collègues membres pour les distinctions de l'ICI au siège social de la SCG d'ici le **15 juillet 2015**.

Les candidatures doivent inclure:

5. un formulaire de candidature de l'ICI dûment rempli qui est disponible sur le site *http://eic-ici.ca/honours_awards/*);
6. une lettre de mise en nomination;
7. le curriculum vitæ du candidat;
8. des lettres de recommandation de collègues, préférablement des fellows de l'ICI.

Les noms des membres de la SCG qui ont déjà reçu des distinctions de l'ICI sont affichés sur le site Web de la SCG à *www.cgs.ca/awards.php?lang=fr*. Il est recommandé que les personnes qui soumettent des candidatures examinent les détails et les critères des prix et médailles avant de les préparer. Pour obtenir de plus amples renseignements, communiquez avec le siège social de la SCG à:

La Société canadienne de géotechnique
8828 Pigott Road
Richmond, C-B
V7A 2C4, Canada
Tel: (604) 277 7527;
1 (800) 710-9867;
Télécopieur: 604-277-7529
Courriel: *cgs@cgs.ca*

Upcoming Conferences and Seminars

68th Canadian Geotechnical Conference
7th Canadian Permafrost Conference
September 20 – September 23, 2015, Québec City, Québec

The **Eastern Quebec Section** of the Canadian Geotechnical Society and the **Canadian National Committee for the International Permafrost Association (CNC-IPA)**, invite you to **GéoQuébec 2015**, for the joint 68th Canadian Geotechnical and 7th Canadian Permafrost Conference. The conference will be held from September 20 - 23, 2015 in the Convention Centre in Québec City, Québec. It will cover a wide range of topics, including speciality sessions that are of local and national relevance to the fields of geo-engineering, permafrost and engineering geology. In addition to the technical program and plenary sessions, the conference will include a complement of workshops, short courses, technical excursions and local tours.

The official languages for the conference will be English and French. The Convention Centre is located in the historic downtown area of Québec City, a UNESCO World Heritage Site, facing onto Québec's Parliament Hill. Old Québec City, which is the cradle of French civilization in North America, is best explored on foot and September is the best time of the year with a typically warm, dry weather and the maple trees just beginning to take on their colourful fall foliage.

The conference theme **Challenges from North to South,** reflects the diverse and complex challenges that the geotechnical, cold regions engineering and permafrost communities will need to address in order to support sustainable economic development. The Local Organizing Committee invites members from the Canadian and international communities to contribute papers on their recent research and advancements in geotechnical, geo-environmental and cold regions engineering, as well as permafrost science.

For more information regarding sessions, topics and the technical program, please visit the web site *www.geoquebec2015.ca* or contact **Jean Côté** (Conference Co-Chair - geotechnical) at *jean.cote@geoquebec2015.ca* or **Michel Allard** (Conference co-Chair - permafrost at *michel.allard@geoquebec2015.ca*.

68e conférence canadienne de géotechnique 7ième conférence canadienne sur le pergélisol 20 - 23 septembre 2015, Québec, Québec, Canada,

La Société canadienne de géotechnique (SCG), la Section régionale de l'Est-du-Québec de la Société canadienne de géotechnique et le Comité national canadien de l'Association internationale du pergélisol (CNC-AIP) vous invitent à participer à GéoQuébec 2015; il s'agit de la 68e conférence canadienne de géotechnique et de la 2e conférence conjointe SCG/CNC-AIP sur le pergélisol. Cet événement se déroulera au Centre des congrès à Québec (Québec), Canada, du 20 au 23 septembre 2015. Le thème de GéoQuébec 2015 – Des défis du Nord au Sud - reflète la diversité des défis complexes auxquels font face les spécialistes en géotechnique, en géotechnique des régions froides et en pergélisol pour assurer le développement durable des communautés canadiennes. Les langues officielles de la conférence sont le français et l'anglais. Le Centre des congrès se trouve à quelques pas du quartier historique de la ville de Québec, un joyau du patrimoine mondial de l'UNESCO, et fait face à la colline parlementaire de Québec. Le mois de septembre à Québec est le meilleur moment de l'année, avec une température clémente et des érables qui se parent de leur feuillage coloré.

Le Comité local d'organisation de la conférence invite les membres des communautés canadiennes et internationales en géotechnique, en géotechnique des régions froides et en pergélisol à contribuer à la conférence en soumettant les résultats de leurs travaux et découvertes dans ces domaines. La conférence couvrira un large spectre de thèmes incluant des séances spéciales d'intérêt local et national dans les domaines de spécialisation de la géo-ingénierie, du pergélisol et du génie géologique. En plus du programme technique et des séances plénières, la conférence comprendra des ateliers, des cours intensifs, des excursions techniques et des visites guidées.

Pour plus d'information sur les sessions, les sujets et le programme technique, visitez le site web *www.geoquebec2015.ca* ou contacter **Jean Côté**, Coprésident de la conférence (géotechnique) *jean.cote@geoquebec2015.ca*, **Michel Allard**, Coprésident de la conférence (pergélisol) *michel.allard@geoquebec2015.ca*.

11th Canadian Conference on Earthquake Engineering July 21 – July 24, 2015, Victoria, British Columbia

The **11th Canadian Conference on Earthquake Engineering** will be held July 21-24, 2015 in Victoria, British Columbia. The conference theme is "**Facing Seismic Risk**", encompassing seismic hazards, engineering, societal planning and response, facility performance, codes and standards.

The west coast of British Columbia is part of the Pacific Ring of Fire, one of the most earthquake prone regions in the world. We are at the collision point of two of the largest tectonic plates on the planet. The risks of a major damaging earthquake along BC's west coast are greater than anywhere else in Canada as we face seismic hazards from four distinct sources. Shallow crustal and deeper sub-crustal events are our most frequent occurrences along with regular events from the Queen Charlotte transform fault. We are also facing the next megathrust from the Cascadia Subduction Zone which can cause a subduction earthquake as powerful as those that recently struck Chile and Japan with the attendant tsunami. These earthquakes could potentially cause severe damage and loss of life.

The focus of the 11th Canadian Conference on Earthquake Engineering is on identifying the risks we are facing, sharing the latest research and advances in seismic engineering and knowledge and encouraging and facilitating engineering practice that will enable us to prepare, protect, mitigate and recover. Please join us in beautiful Victoria, the capital city of British Columbia, located on Vancouver Island. Experience nature and networking, engage with colleagues and locals, and leave feeling inspired and informed. For more information, visit the conference website *www.canadianequakeconference.ca or secretariat.ccee@caee-acgp.ca.*

Heritage Committee

History of Local Chapters of the Canadian Geotechnical Society

The Heritage Committee believes that the history of the local chapters 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 chapters. Hopefully these stories will encourage other local chapters of the CGS to gather their archives and write their own history.

If you have any questions or have other historical information that you wish to share or know of any opportunities to acquire material that is at risk of being lost, please contact the Chair of the CGS Heritage Committee, **Suzanne Powell**, **P.Eng**., at *spowell@thurber.ca*

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Introduction by John Dunnicliff, Editor

This is the 82nd episode of GIN.

One article this time, a discussion and a closure.

Use of the ShapeAccelArray (SAA) in a rockfill dam

The article by Marc Smith compares settlement data collected from an SAA with data collected from a conventional horizontal inclinometer during the recent construction of a rockfill dam. There is clear preference for the SAA.

Because data collection will be ongoing, the author has agreed to send me a contribution for GIN in about three years' time, to update us on the accuracy and durability of the SAA, by which time it will have been in place for about six years.

Discussion and closure of article in December 2014 GIN about wireless monitoring

The discussion by Adam Dulmage and Matt Trenwith of "The fundaments of wireless monitoring – things to consider" by Simon Maddison is followed by a closure by the author.

Another corporate update

In March 2015 GIN I reported on several corporate changes, notably the acquisition by Nova Metrix LLC, Woburn, MA (www.nova-metrix.com) of various instrument manufacturers with familiar names. Nova Metrix has now acquired Schlumberger Water Services Technology Group, which is comprised of Westbay Instruments and Waterloo Hydrogeologic. Those two companies will be familiar to GIN readers as manufacturers of multipoint piezometers.

What are the characteristics of an engineer?

An astronomer, a physicist, and an engineer were travelling north from London by train. They had just crossed the border into Scotland, when the astronomer looked out of the window and saw a single black sheep in the middle of a field. "All Scottish sheep are black," he remarked. "No, my friend," replied the physicist, "*Some* Scottish sheep are black." At which point the engineer looked up from his paper and glanced out of the window. After a few seconds thought he said blandly: "In Scotland, there exists at least one field, in which there exists at least one sheep, at least one *side* of which is black".

Closure

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

Eis Igian (Greece)

Performance of a ShapeAccelArray (SAA) for settlement monitoring of a large rockfill dam

Marc Smith

Introduction

A ShapeAccelArray (SAA, www.measurandgeotechnical.com) was installed alongside a conventional horizontal inclinometer (INH) during the recent construction of a dam. This setup allowed the comparison of settlement results from both types of instrument and helped gain confidence in the relatively new SAA technology for embankment dam engineering. This article shows Hydro Québec's (Canada) experience with the performance of a SAA used to monitor settlements in a large rockfill dam during its construction. This experience is based on a dam safety context where instrumentation is permanent and expected deformations are relatively small and progress slowly.

Dam cross section and instrumentation

The Romaine-2 dam is a 112 m-high asphalt core rockfill structure part of the Romaine-2 hydroelectric project located in northern Québec, Canada. Dam construction took place mainly in 2012 and 2013 after river diversion by means of two cofferdams. Reservoir impoundment started during spring 2014.

The asphalt core has a width varying from 0.85 m at its base to 0.5 m near the crest. It is flanked on both sides by support and transition zones (3M and 3N) having maximum particle sizes of

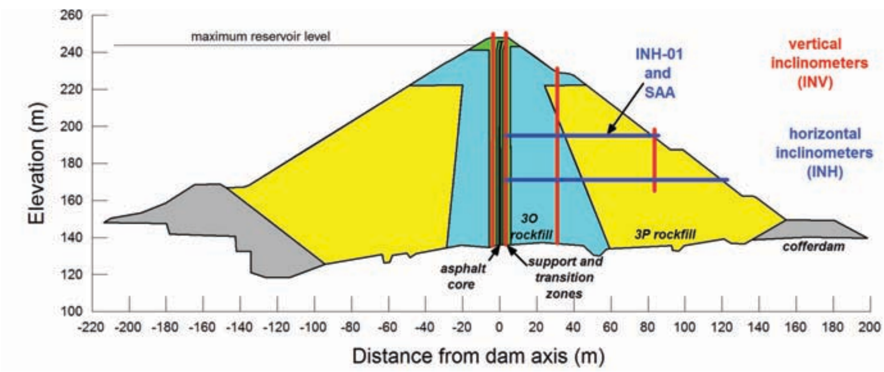

Figure 1. Schematic cross section of the Romaine-2 dam and location of inclinometers and SAA.

80 mm to 200 mm. The rockfill shell is comprised of two zones: the 3O internal shell has a maximum allowable size of rock particles of 0.6 m compared to 1.2 m for the 3P outer shell. Figure 1 shows a schematic cross section of the dam at valley center.

Material placement procedures were of utmost importance to prevent excessive fill movements during dam construction and operation which could have detrimental effects on the thin asphalt core. The placement of support/transition as well as rockfill zones required optimized material characteristics and increased compaction energy to achieve maximum density and thus minimize settlements during construction, impoundment and operation. Therefore, internal deformations of the dam needed to be closely monitored to assess its behaviour as well as in situ materials rigidity parameters to be used for stress/deformation modelling and also to quantify the effects of the increased compaction energy used for the Romaine-2 dam compared to other Hydro Québec projects.

A series of inclinometers is installed in the dam body to measure deformations (see Figure 1). A total of four vertical inclinometers (INV) anchored in bedrock (far end considered fixed) are used to monitor movements closer to the core as well as in the 3O and 3P rockfills. The INV in the 3P zone represented on Figure 1 is located at a section where bedrock elevation is higher. Two horizontal inclinometers (INH) and one ShapeAccelArray (SAA) are also installed to monitor settlements. The far and near ends of these three instruments are not considered fixed. Figure 2 shows the location of the INV, INH and SAA.

The SAA is installed along INH-01 (see also Figure 1). An access road on the dam crest and downstream face allows instrumentation readings.

INH characteristics

The two INH are composed of 1.5 m-long grooved ABS casings installed horizontally in a trench excavated in the placed rockfill. Settlement readings are made using an accelerometer probe which measures tilt at every 0.5 m in the plane of the probe wheels travelling in the top and bottom grooves of the casings. The probe is inserted in the horizontal inclinometer using a system of return cable and pulley. The return cable is installed within a separate pipe alongside the inclinometer casing. The tilt measurements from two sets of readings (probe reversed end-for-end) are converted to settlements at the office.

INH were installed in other Hydro Québec projects but have been subject to operation problems after two to three years due to ice build-ups inside the casings as well as pulley and return cable malfunctions. These problems had a significant effect on the availability and the reliability of results.

Long-term settlement monitoring along a horizontal plane gives valuable information related to the deformation of the various types of materials constituting an embankment dam. Deformations need to be measured during the construction (load increase due to fill placement), impoundment (load due to reservoir) and operation (creep) phases of the dam life cycle. Another option was thus needed to obtain reliable settlement measurements. A SAA was therefore installed in the Romaine-2 dam to gain confidence in this relatively new technology.

SAA characteristics

A SAA consists in a series of rigid segments separated by special joints which can tolerate the range of settle-

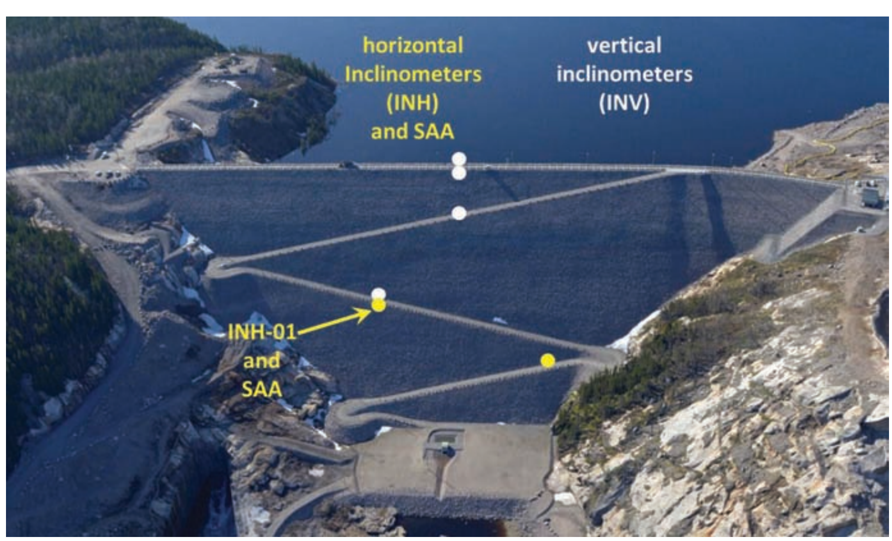

Figure 2. Location of inclinometers and SAA.

Figure 3. Installation of INH-01 and SAA.

ments expected in the dam. Each segment contains a triaxial gravity sensor measuring tilt at every 0.5 m which is automatically converted in settlements. The readings are made using a portable computer and a specialized program. The Romaine-2 SAA is located next to INH-01 to compare results from these two types of instrument. Figure 3 (looking upstream) shows the installation of INH-01 as well as the SAA which needs to be inserted into a protective PVC conduit.

The installation procedures for both types of instrument are similar. They have to be placed in an excavated trench and protected from large fill particles by using bedding sand, geotextiles and by controlling the grain size distribution of surrounding soils. Moreover, as for a INH, twisting of the SAA must be avoided by carefully aligning cable markers since the software used for data collecting and processing is calibrated according to this alignment.

Measurement and data processing procedures

Measurements along the Romaine-2 SAA (and also INH-01) are taken relative to the near end (the downstream end of the instruments i.e. near the operator) and are corrected considering measured displacements of a nearby survey point.

Readings of the 76 m-long INH-01 requires at least two persons for the handling of bulky equipment and cables. Vehicle accessibility to the instrument is thus essential. Reading time is in the order of hours and can be greatly increased in adverse weather conditions which can also decrease measurement reliability. Only basic checks of the reasonableness of readings can be made in the field. Data processing programs needed to be customized for the Romaine-2 context. Moreover, the INH probe was subject to bias shift errors for which corrections were not trivial since both ends of these instruments are not considered fixed.

The Romaine-2 SAA has a length of 76 m. The specified maximum instrumented length of a typical SAA cable is 100 m but multiple cables can be joined to allow measurements for greater lengths. Readings and data processing are realized in minutes using a portable computer. The actual shape of the series of SAA segments can be immediately viewed on screen. Automated data acquisition and transmission are also possible which can alleviate instrument accessibility problems in Romaine-2 such as in winter when the downstream face of the dam is covered with snow.

Reported accuracy of instruments

The reported random error for INH measurements is approximately ± 1.4 mm per fifty readings. Considering that this type of error tends to accumulate with the square root of the number of readings, the expected random error for INH-01 would be around ± 2.4 mm. However, systematic errors such as those related to probe bias, depth positioning and the effects of adverse weather on the instrument (and the operators) can be much higher and cannot always be entirely corrected.

The reported accuracy deformation value for a SAA is ± 1.5 mm per 32 m. This value tends to increase with the square root of the length which leads to an accuracy of ± 2.3 mm for the SAA installed in the Romaine-2 dam. This value has been confirmed

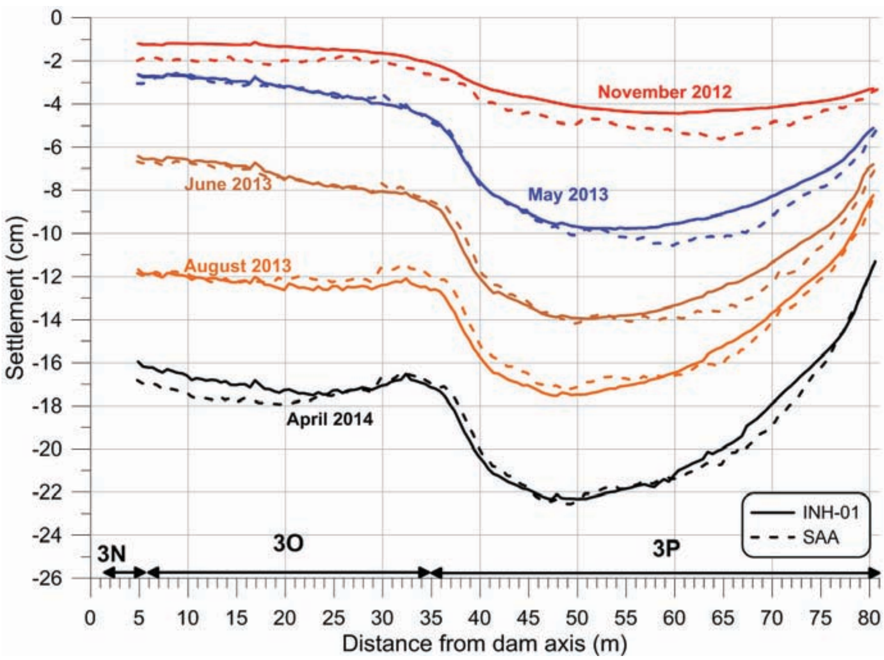

Figure 4. Measured settlements by INH-01 and SAA.

occasionally in the field by repeated measurements for time intervals of less than a few days. Also, the effects of systematic errors, if present, have not been identified.

The random errors/repeatability of both types of instrument are comparable. They are adequate for the purposes of settlement measurements in Romaine-2 namely dam safety assessment and modelling as well as quantification of the effects of increased compaction energy. The uncertainties related to other geotechnical parameters pertaining to the dam have a greater influence on these three aspects. Differences in global accuracy between the SAA and INH-01 are mainly due to systematic errors related to the reading conditions and data processing procedures.

Measured settlements in the Romaine-2 dam

Installation of INH-01 and SAA took place in October 2012 (see also Figure 3). This date corresponds to the initial state of the instruments from which subsequent readings are compared to compute settlements. Figure 4 shows a sample of measured settlements during the construction phase of the Romaine-2 dam.

Both types of instrument clearly show a greater compressibility of the 3P rockfill, as expected. April 2014 corresponds to the last reading during the construction phase. This date now corresponds to the new initial state for the impoundment and operation phases in which the settlement measurements are carried on.

Results on Figure 4 also show that differences between INH-01 and SAA are less than ± 1 cm which is acceptable considering the dam height and thus the internal stresses (up to 2 MPa) and also, as stated before, the uncertainties related to other geotechnical parameters. Figure 5 shows a more detailed representation of these differences. A positive difference indicates that INH-01 measured a greater settlement value than the SAA.

Differences shown on Figure 5 are representative of random errors and uncorrected systematic errors pertaining to INH-01 and SAA.

Conclusions

Settlement monitoring of the Romaine-2 dam is required during the construction, impoundment and operation phases of the dam life cycle. The analysis of the internal deformations allows the assessment of dam behaviour as well as in situ materials rigidity parameters for stress/deformation modelling. The effects of increased compaction energy used during construction can also be quantified. However, the uncertainties related to

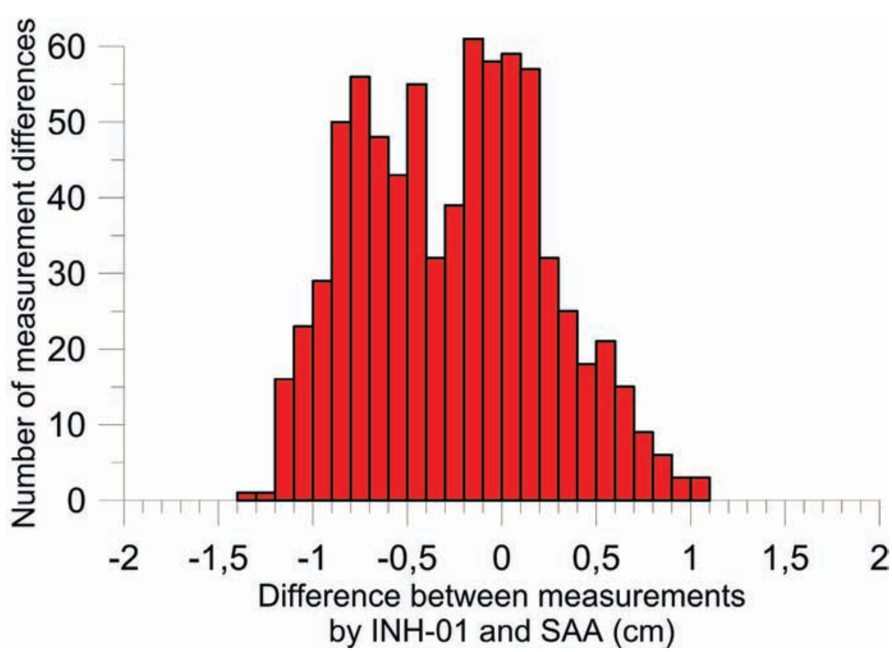

Figure 5. Differences between measured settlements by INH-01 and SAA.

other geotechnical parameters pertaining to the dam have a greater influence on these three aspects than the differences between the SAA and INH-01 measurements.

Considering this context, a SAA represent an interesting alternative to a conventional INH. The installation procedures for both types of instrument are similar as well as global accuracy although the SAA appears less prone to systematic errors. However, the SAA provided significant advantages over INH-01 due to easier and faster measurement, in situ checking and data processing procedures. Simple automatic data acquisition and transmission options are also available for the SAA which can alleviate accessibility problems and give more flexibility in determining instrument reading frequency.

Both INH in the Romaine-2 dam began to show signs of malfunctioning after less than two years of operation. Operation of the pulley and return cable became more difficult with time, and ultimately impracticable due to excessive probe and cable friction inside the inclinometer casing and/or the return pipe. Readings had to be postponed until summer 2015 to assess if these friction problems are caused by ice build-ups. The SAA is still performing well after nearly three years but its long-term durability and accuracy remain to be proven.

SAA offers more possibilities than conventional inclinometers for measuring internal deformations in dams, since there are no series of casings to install and to access later for readings. A series of six horizontal SAA cables will be installed in a 92 m-high embankment dam to be constructed in 2015 and 2016 to monitor settlement in the upstream and downstream rockfill shoulders

The Romaine-2 experience has shown that a SAA installation can have higher initial hardware costs than a conventional inclinometer. However, these costs can be recouped in a longer term considering reading and data processing time, instrument accessibility as well as durability.

Both types of instrument, SAA and INH, provided useful results for Romaine-2 but the SAA did so more conveniently, with more flexibility and, apparently, for a longer period of time.

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Discussion of "The fundaments of wireless monitoring – things to consider" by Simon Maddison. Geotechnical News, Vol. 32, Number 4, December 2014

Adam Dulmage and Matt Trenwith

This is a very useful article when considering data acquisition options for geotechnical monitoring (or any application for that matter). We have direct experience with mesh networks in mining environments, primarily underground, but also many surface applications, and we will touch on some of the lessons learned in these harsh environments.

The term 'wireless'

In many cases, the term 'wireless' is used interchangeably with 'Wi-Fi' - so let's clarify this point first (as this tends to be a hot topic with mining companies right now). 'Wireless' can be any type of technology that does not use wires for communication. It can use any range of frequencies, bandwidth, protocol, antenna type, etc. It is a very generic term. 'Wi-Fi' is much more specific and is defined as any wireless local area networking product based on the IEEE 802.11 standard. This is what most home wireless networks are built upon - your computer and your cell phone typically have a Wi-Fi radio built into them. 'Wireless' as it relates to geotechnical monitoring is almost always NOT Wi-Fi, but often a purpose built sensor network designed just for data acquisition and monitoring of (typically) low power sensors.

Frequency selection

So, onto the good stuff. Talking about frequencies — 2.4GHz is generally license-free worldwide, and 900MHz is license-free primarily in North America and Australia, so this needs to be considered at the beginning of the project. However, additional restrictions may be imposed by the mining firm, especially in blasting zones. There is also a significant difference in signal propagation between 900MHz and 2.4GHz. 900MHz is more forgiving, allowing non-line-of-sight

(NLOS) transmission which is often the case for an underground or tunneling environment where line of sight can be challenging. Another consideration is power consumption and range. All other things equal, a 900MHz radio will provide up to 2.7 times the range than the otherwise equivalent 2.4GHz product for the same given transmit power. This means that for a same given installation the transmit power of the 900 MHz radio can be reduced, further improving battery life. A typical underground range for our 900MHz mesh network (battery-powered) is between 50-150m at +14dBm transmit power and using a +3dBi omnidirectional antenna , but can sometimes throw as far as 350m when tunnel size and conditions are ideal. Surface range with a standard omni-directional antenna is typically 300-1000m.

Data backhaul

Data backhaul options in mining are unique in comparison to surface. Often there are no backhaul options at all, and in these cases a store-and-forward type of system where the data are collected and relayed to a central gateway which can then be polled at a later time is quite beneficial. However, this is not the ideal option as real time data is sacrificed. In most North American mines there is usually a radio network for voice communication (called Leaky-Feeder), and also fibre-optic cables for backhaul. Fibre is always the preferred option, allowing for much higher bandwidth than leaky feeder, and in most cases this is what is used in the top-tier mines worldwide. In this way, the mesh is deployed to the point where the sensors are installed (sometimes upwards of 50 nodes in a linear fashion) and relayed back to the gateway for backhaul over fibre to surface.

Network topology

The network topology should always be designed with robustness in mind, so ensuring that there are redundant links is important. In the case of mining, wireless node placement is critical to ensure not only that the signal propagation is the best available, but the risk of damage to the node is minimal from effects from blasting and damage caused by vehicles. The considerations for surface deployments are often very different in nature. Things like snow, rain, wind, extreme hot or cold then become potential issues and cause for concern. Snow and rain may affect signal propagation, whereas with extreme hot or cold one also has to consider the effects on battery life over time. If you deploy in the middle of winter in a forested area, what will be the effect of leaves growing on the trees in the spring - will this affect your signal (let me answer that for you: yes). If careful consideration is taken during the planning phase, there is a very high probability of success during deployment.

The future of wireless monitoring is promising, and should never be discounted just because someone has ‘tried wireless before’ without success. Both businesses and consumers alike are driving the research and development of new wireless technologies and applications every day, so what may have been problematic before can now be resolved. It's always worth picking up a copy of GIN to see what's new and improved!

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Response/ Closure

Simon Maddison

It is very positive to hear of Adam and Matt's practical experiences with using wireless mesh for monitoring in the extremely demanding and specialised domain of mining. This is precisely the sort of circumstance where mesh shows its strengths in terms of ease of deployment, robustness and flexibility – but properties that are also indispensible in many if not most geotechnical monitoring applications.

They make some very valuable points relating to wireless frequency and power. There are limitations on certain frequencies in many countries, as well as specific radiated power limits, both factors which are generally treated much more liberally in North America in comparison to Europe for example! This is a challenge for suppliers operating in international markets in terms of what equipment operating frequency and power is supplied to reach the largest possible range of customers. For this reason 2.4GHz is probably the most favoured frequency.

For data backhaul, it is correct in our experience that it is necessary to work with whatever options are available when underground. However with a flexible gateway solution, it should be possible to hook up to whatever transmission media is available, using industrial grade communications interface equipment. We have provided a multiplicity of such solutions for a range of installations in metro railway tunnels, including the use of solid state industrial PC's for storage and even rendering of data for local access.

My final point is that there are a number of emerging wireless monitoring companies, often with claims that cannot be backed up or where performance is not as stated. I fully endorse the conclusion regarding wireless geotechnical monitoring solutions, but go further. Wireless should be a prime choice but only one that has been shown to really work; then and only then can one say there are now available leading-edge solutions supporting 100+ node networks and running for up to 15 years on a single battery reporting every 20 minutes and with stable precise data in a tough and busy mining or geotechnical environment.

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The Value of Failure

G. Ward Wilson

Andrew MacG. Robertson

A breach occurred within the perimeter embankment of the Mount Polley Tailings Storage Facility on August 4, 2014. The loss of containment was sudden and occurred without warning. An Independent Expert Engineering Investigation and Review Panel (IEEIRP or panel) was quickly commissioned by the British Columbia Ministry of Energy and Mines. The IEEIRP consisting of Norbert Morgenstern (Chair), Stephen Vick, and Dirk van Zyl released their report on the Mount Polley Tailings Storage Facility Breach on January 30, 2015.

The media and public interests

Reactions in the media were as expected with statements such as those of Steven Hume at the Vancouver Sun who on February 11, 2015, wrote, "It is now pretty clear what happened at Mount Polley leading up to the dam bursting last August and spilling 24,000,000 m^3 of toxic mine tailings, silt and waste water into the Quesnel, and ultimately the Fraser River systems, potentially putting thousands of people and millions of migrating salmon at risk". No matter how theatrical and spectacular we think such reports are, they capture the paradigm of the culture within which we must operate and ultimately obtain social license.

Fortunately, no loss of life occurred as a result of the release and since the quality of the water that was released was actually quite good, the size of the fish kill was much smaller than the proclamation above would have us believe.Nevertheless we must recognize the serious implications of the event at Mount Polley as it is one of the most significant event in the professional and corporate memories for many of us engaged in the business of mine waste management. While it is an event we consider to be of extremely high consequences, it may well be a blessing in disguise. It could have been much worse in terms of loss of life, environmental damage and financial cost if the event occurred at another site. There are a number of significant tailings impoundments in close proximity to large urban Canadian populations. We recall the failure that occurred in 2012 at the bauxite mine at Kolontar, Hungary, that inundated several towns, killed ten people and flooded more than 8 km^2 of the surrounding terrain.

Possibly the greatest lasting consequence of this failure is the breach in trust that has occurred in the reliability of modern tailings dams constructed by responsible companies in a well regulated jurisdiction, under conditions which were not extreme. Perhaps the greatest benefit that can come from this is the great and lasting change that is occurring in the mining industry world wide, as engineers responsible for the design and operation of tailings dams, and the corporate and regulatory leaders responsible for the governance of such structures, recognize their fallibility and strengthen the processes needed to ensure that they are not contributors to the next failure.

The Province of Alberta has some of the largest tailings dams in the world. In his keynote address to the Tailings and Mine Waste 2010 Conference delegates in Vale, Colorado, Dr. Morgenstern stated that Syncrude's out-of-pit tailings pond has a perimeter of about

Greens Creek filtered tailings stack showing drainage capping and progressive reclamation.

18 km and may well be the largest earth structure in the world in terms of volume of engineered fill. In that same address, Dr. Morgenstern also quoted the internationally respected water ecologist Dr. David Schindler who declared, "If any of those tailings ponds were ever to breach and discharge into the river, the world would forever forget about the Exxon Valdes".

The writers believe that the dam safety systems for the oil sands in the Province of Alberta are of the best worldwide. In contrast to this opinion, the most recent report of the Auditor General states, "The department's dam safety group has no requirement to document its work, and without such reports, it's hard to know if Alberta's dams are safe". One could argue that the greatest value found in any one failure is that it rivets attention and forces us to examine and re-evaluate all of our structures and systems.

The report

The report on the Mount Polley Tailings Storage Facility Breach issued by the IEEIRP is comprehensive. The principal finding was that the breach within the perimeter embankment occurred as a result of a foundation failure in a glaciolacustrine layer referred to as the Upper GLU. The full panel report can be found at *https://www.mountpolleyreviewpanel.ca/final-report*. We encourage our readers to download and examine the full content of the report.

We cannot discuss the details of the panel report nor all of its discussions, recommendations and conclusions in this brief article. However, one of the most significant statements is centered on the historical record of active tailings dams in the province of British Columbia during the 46 years since 1969, and the seven failures that have occurred during this period. Statistical evaluation of the historical record finds a failure frequency of 1.7×10^{-3} per dam per year or approximately a one in six hundred chance of a tailings dam failure in any particular year. In other words, without improved performance, the province can expect (on average) that there will be two failures every 10 years. Based on this analysis, "the panel firmly rejects any notion that business as usual can continue".

With respect to risk-based dam safety practice, "the panel does not accept the concept of a tolerable failure rate for tailing dams". This assertion resulted in the panel recommending the implementation of the best available tailings technology (BAT) based on the BAT principles that are outlined as follows:

1. Eliminate surface water from the impoundment.
2. Promote unsaturated conditions in the tailings with drainage provisions.
3. Achieve dilatant conditions throughout the tailings deposit by compaction.

While the panel recognized the issue of chemical stability associated with the elimination of water from the tailings deposits, the BAT principles stand as a strong recommendation for the future of tailings management. Implementation of the BAT principles for the surface storage of tailings can lead to the use of filtered tailings technology. Filtered tailings technology, often called "dry stack tailings", when properly designed and formed can satisfy each of the BAT components. The panel points to the Greens Creek mine in Alaska as an example where "dry stack tailings" have been successfully constructed in a wet climate that is similar to many sites in British Columbia.

Implications

The first point we would like to remind our readers is that the report on the Mount Polley Tailings Storage Facility Breach and its recommendations were written for the Government of British Columbia and outline actions to ensure that similar failures do not occur at other mine sites in the province. For clarity, it is important to point out that the mandate given to the panel was for the 'Safety Case' and not the environmental issues related to chemical stability. Thus the panel points out "water covers run counter to the BAT principles" and that "the

Greens Creek filtered tailings stack showing trucked tailings placement and trafficability.

Mount Polley failure shows why physical stability must remain foremost and cannot be compromised". We also recognized that the case of 'filtered stacked tailings' at Greens Creek was given by the panel as an example for the application of BAT principles, and that other methods of tailings disposal can achieve these principles and should also be evaluated. Adoption of the BAT principles will create new challenges as well as new opportunities for the designers of new surface tailings impoundments. The writer's note that in the specific case of 'dry stacking of filtered tailings' this method should not be considered a panacea for the elimination of failure potential. One of the writers is currently reviewing two such dry stacks where the design, operating and site conditions have lead to an urgent need for remedial modifications to avoid failure conditions.Furthermore, the implementation of BAT principles for physical stability (BAT-PS) is not necessarily BAT for chemical stability (BAT-CS). The tradeoff in risk is real and must be addressed in light of all stages of environmental assessment. We note that on March 19, 2015 the Environmental Assessment Office of the British Columbia Ministry of Environment released an Information Bulletin „Province Implements Post-Mount Polley Requirements in Environmental Assessment" outlining information that companies proposing to build new tailings dams will be required to include. For example, selected options for tailings management will need consider other options that address the potential for adverse effects on the environmental as well as on health, social, heritage and economic issues. Furthermore, companies will need to present and compare best practices and best available technologies for tailings management.

Having just attended the 10th International Conference on Acid Rock Drainage (ICARD), it is apparent many geochemists will resist the implementation of the BAT-PS principles. Geochemists dedicate themselves to chemical stability, as opposed to physical stability, for the prevention and control of acid rock drainage and metal leaching (ARD/ML). A key design principle for the prevention of ARD for tailings is to maintain full saturation within the profile, and in many cases water covers are used to prevent surface oxidation. Water covers are routinely employed as a preferred strategy for the long-term closure of reactive tailings in Canada, a practice that has been embraced worldwide as outlined by the global acid rock drainage (GARD) guide. In summary, the general principles laid out in the GARD guide (BAT-CS) contradict the BAT-PS principles, leaving mine waste professionals faced with competing or conflicting design criteria. The IEEIRP, along with geotechnical engineers and geochemists recognize this "Catch 22" situation.

The province of British Columbia and its technical community of experts in ARD/ML are world renowned. British Columbia aquatic standards are often used as objectives for water management by mining companies worldwide. The tradition of expertise in British Columbia dates back several decades as can be seen in the publication of the Draft Acid Rock Drainage Technical Guide that was prepared for the British Columbia Acid Mine Drainage Task Force in 1989. Furthermore, the Annual BC-MEND ML/ARD Workshop has enjoyed more than 20 years of success, regularly attracting experts from around the world. We do not want to overstate the expertise and capacity in British Columbia to address and manage the chemical stability of reactive mine rock, but it is certainly world class. Furthermore, we believe the expertise available within the technical community of British Columbia will be successful in the integration of the BAT-PS principles with the well-established principles for the prevention of acid rock drainage and metal leaching (BAT-CS).

New opportunities

The development of new technologies and improved methods for the management of reactive mine tailings lie ahead of us. A key component for the design of dry tailings stacks will be the degree of water saturation within the tailings profile established over the long term. For example, tailings profiles with water saturation levels less than 85% are considered resistant to liquefaction. Conversely, tailings profiles with water saturation levels greater than 85% are considered resistant to oxygen diffusion and subsequent ARD. Successful implementation of the BAT principles for both PS and CS will now rely on a comprehensive understanding of unsaturated soil mechanics. The Soil-water characteristic curve (SWCC) of the tailings controls water saturation as a function of matrix suction (i.e. negative porewater pressures) along with the unsaturated hydraulic conductivity of the tailings. The design of cover systems to control infiltration rates and oxygen fluxes for the prevention and control of ARD in reactive mine tailings is based on coupling the hydraulic properties of the SWCC with microclimatic conditions associated with precipitation and evapotranspiration. This technology is considered well developed and can be extended to predict the unsaturated hydraulic performance of dry tailings stacks. For example, the design of the cover system used for final closure of the stack will need to provide infiltrative fluxes that optimize unsaturated flow conditions in the tailings.

De-sulphurization and the production of clean tailings will also provide new opportunities. De-sulphurized tailings were applied as clean cover for closure of the tailings beach at Detour Mine in 1999 (then owned by Placer Dome). A recent study reported by Cash et al. (2012) demonstrated that that cover has performed very well more than 10 years since closure. A residual sulphur content of less than 0.5% was allowed to remain in the tailings so that the cover profile (typically 1.5 m thick) would be non-acid forming and oxygen consuming. The details for the design and the predicted performance of the de-sulphurized cover at Detour are fully described by Dobchuk et al. (2013).

The design of dry tailings stacks for both operation and closure will advance as the BAT-PS principles are coupled with the BAT-CS principles for the prevention and control of ARD/ML. The use of filtered tailings and dry stacks will reduce (not eliminate) the risk of failures such as we have seen at Mount Polley, but at the same time will create new risks that will need to be managed. For mine sites in the province of British Columbia, ARD/ML will likely become one of the most significant risks for long-term environmental impacts and closure costs. New methods of risk assessments to optimize the design of stacks that maintain the best combination of BAT-PS and BAT-CS principles and requirements will be necessary. There is extensive experience in British Columbia and internationally in tailings impoundment design with risk assessment using failure modes and effects analyses (FMEA). FMEA does not eliminate risk, but is a tool that allows identification and quantification of risk and therefore the selection and application of mitigation measures to reduce risk. Shaw and Robertson (2015) demonstrate that FMEA methodology can be applied also to assess the risk of ARD/ML and to identify mitigation measures to better manage uncertainty and errors associated with ARD/ML prediction and control. By including both PS and CS failure mechanisms, likelihood and consequences in the FMEA, one is better able to optimize the tradeoffs between PS and CS measures and therefore minimize overall risk. Surely the minimization of overall risk is what we strive for in defining 'Best' in BAT.

In the view of the authors, filtered tailings dry stacks are not the only method for BAT-PS. We are certain the panel is simply not recommending the use of filtered tailings dry stacks, but more importantly they are recommending BAT Principles. Alternatives may include paste and thickened tailings, cycloned sand or even conventional slurries with extended beaches, underdrains and compacted lifts. Similarly, wet disposal of ARD tailings should not be considered the only method of BAT-CS. Rather BAT is that combination of technologies, that when combined, result in the least risk of physical and chemical instability that could potentially lead to failure. BAT may include filtered dry stacked tailings or wet disposal of ARD tailings if their inclusion in the structure results in least overall risk.

A transition to filtered tailings, where appropriate, may also offer additional new opportunities for mine waste management. An immediate benefit may be the reduction of the footprint required for the tailings impoundment. Progressive closure with construction of the stack should also be possible. Co-disposal of reactive mine waste rock, for which ARD can be very difficult to prevent and control, may be possible. For example, reactive waste rock layers may be co-mingled with filtered tailings to serve as sealing layers that prevent advection of oxygen in the waste rock. Vertical ribs of waste rock may also be constructed in the profile of the filtered tailings stack to improve drainage and stability. In addition, filtered tailings may be blended with waste rock and mixed at an optimum ratio to form a dense high strength paste rock.

Scale may also present of the additional challenges and difficulties in the implemention of BAT-PS and CS principles. The implementation of filtered dry stacked tailings at large mines may be prohibitively expensive or require logistics that we are not yet capable of managing. While the technology and capacities for filtration systems are rapidly developing, metal mines keep getting bigger with tailings production rates often exceeding 120,000 tpd, and the largest currently under construction at 360,000 tpd. We now have a number of tailings dams under construction with ultimate toe to crest heights exceeding 300 m, and the highest exceeding 400 m. The elimination of ponds may be difficult in very wet climates where runoff control has to be practiced, and the construction of stacks may be difficult in high rainfall regions. Clay-rich tailings may prove difficult to dewater by filtration sufficiently to construct and maintain stable stacks. In many cases, BAT for these mines may not include filter pressed stacked tailings, but this does not preclude or lessen our need to strive for BAT that includes the most optimum combination of technologies needed to reduce risk to socially acceptable, very low values.

Conclusions

We see value arising from the failure at Mount Polley. We expect the drive to evaluate, test and implement emerging and new technology will add knowledge that will generate opportunities for improved methods of design, construction, monitoring and regulation that will create earth structures on mines that are both physically and chemically stable. These new earth structures will be easier to reclaim and transition to land uses that are environmentally secure and socially acceptable.

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IN MEMORIAM

Nguyen Truong Tien
1950 – 2014

It is with sadness that I received word of the passing of Dr. Nguyen Truong Tien on October 16, 2014. Dr. Tien was a leader in bringing advances in Soil Mechanics and Geotechnical Engineering to the country of Vietnam.

Dr. Tien started his engineering studies in Construction Engineering at Polytechnic University in Havanna, Cuba in 1967. He spent 9 years in Cuba going on to obtain a Master's degree through a cooperative graduate program with the University of British Columbia. In 1976 Tien accepted a position at the Institute of Building Science and Technology (IBST), a position that he held until 1995. In 1985 Tien was invited to undertake further graduate studies at Chalmers University in Sweden. He completed his doctoral thesis in soil mechanics within 2 years.

In 1991 he established a foundation engineering and construction company known as COFEC. The company became known as a design consulting firm with a reputation for outstanding engineering skills. COFEC also became the means of training many young engineers who went on to pursue masters and doctorate degrees in countries such as Canada, Sweden, Japan, Thailand, Malaysia, Singapore and the United States. Through his unique networking capabilities he brought many benefits to the construction industry in Vietnam. He viewed technical conferences as being extremely useful to the development of Vietnam, thus becoming the chairperson of conferences that invited scientists and engineers to present technical information.

In 1997, Dr. Tien was appointed to the Ministry of Construction as the general director of the Technical Development Company referred to as TDC. Between 2002 and 2010 Dr. Tien worked for the General Construction Company of Hanoi, then retiring from active engineering practice. Subsequently, he became a prolific writer describing his philosophical view for the Vietnamese people.

It was during my first trip to Vietnam in 1993 that Dr. Tien and I began to formulate a CIDA (Canadian International Development Agency), proposal for technology exchange between Canada and Vietnam. The proposal was funded by CIDA and led to almost 10 years of close cooperation and technology transfer between Canada and Vietnam. It was also during my first visit to Vietnam that Dr. Tien proposed the formation of the Vietnamese Geotechnical Society and the publication of the Vietnamese Geotechnical Journal. These two initiatives are examples of the unique abilities of Dr. Tien to "birth" visions for technological advancement and bring them to fruition.

Many challenges were associated with practicing geotechnical engineering in a country with so few financial resources. In the years following my first visit to Vietnam I made visits on an almost annual basis. During those years I developed a close friendship with Dr. Tien and his family. It was Dr. Tien who introduced me to the Vietnamese culture with its warm hospitality and unique way of making a person feel welcomed.

Dr. Tien had a vision for the development of geotechnical, civil and environmental engineering in Vietnam. He shared his visions with me and I always wished there was more that I could do to bring his visions to fruition. I sent used engineering books to Vietnam and he showed immense gratitude and appreciation for my miniscule efforts. Many of his visions for the importance of geotechnical engineering will live on into the next generation. One of the highlights that I will always remember from my collaborative program with Vietnam is the trip of a Vietnamese delegation to Canada in the late 1990s.

I will forever be grateful for all the kindness that Dr. Tien showed to me over the years.

Delwyn G. Fredlund

Paolo Gazzarrini

Overture

39th episode of the Grout Line and for this issue we have an article about a "different" type of grouting written by my good friend Jim Warner, Grouting Consultant. Jim is author of the Grouting Bible: Practical Handbook of Grouting - Soil, Rock and Structure, which, in my opionion, all grouters should have in their library.

The article is not related to conventional grouting in soil or rock, but in structures, mainly concrete and masonry. I agree with Jim that established grouters could make better use of opportunities where structural grouting could be used.

Enjoy your reading!

And, as usual, the same request, asking you to send me your grouting comments or grouting stories or case histories. My coordinates are:

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

Ciao! Cheers!

Grouting in Structures

James Warner

While pressure grouting is most often used in geomaterials, it is also a favorable technology for the repair and retrofit of structural concrete and masonry. While many “geotechnical” grouters turn their back to such applications, there is significant benefit of such work, which involves essentially the same principals and parameters as geotechnical applications, with the exception that significantly smaller quantities are common, and thus smaller, lower capacity equipment. Also, grout stability, shrinkage, and strength become essential considerations.

Figure 1. Here are two defects in one location. Both honeycomb and a crack are visible.

Figure 2. Tight fitting form over honeycombed surface contains grout during injection.

Figure 3. Pressure testing grout hole using line pressure of facility system.

Figure 4. Leak on the surface in an area where no problem was previously visible.

Concrete

The most common voids found in concrete are cracks, which can of course be observed on the surface. Not visible are interior faults which can be in the form of rock pockets (honeycomb) that result from the large aggregate becoming separated from the mortar fraction during placement, or completely empty voids usually resulting from interference of the reinforcing steel or other embedments to the placement flow and vibration compaction during original casting. As in geomaterials, location of internal deficiencies in concrete is difficult, and similar methods such as coring, and seismic wave propagation are used. Where defective zones are confirmed or suspected, providing a grid of regularly spaced grout holes is common.

Honeycomb is easily repaired by simply drilling holes to intersect it or if on the surface, placing a tight fitting form over it as shown in Figure 2. A stable cement grout is then injected so as to fill all the voids. While it is very common practice to simply chip out the porous concrete followed by filling with mortar or drypack, because it is often very difficult to remove all the culprit honeycomb, such does not always provide the best repair. Pressure grouting has the advantage that all of the voids can be filled, so long as sufficient venting of any entrapped air is provided. This is not much of a problem with most concrete as it is typically somewhat permeable to air. Further, it is extremely difficult to construct an air tight form, so ample venting is virtually always available.

Treating non visible voids in the interior of mass concrete is much more difficult in that the exact location of the suspected voids is seldom known, but must be established. This might be by way of coring or non-destructive seismic techniques. In many cases however, voids can be so extensive that a regularly spaced grid of grout holes over the suspect area is best. It is important that the holes not completely penetrate the section however as grout travel must be contained within the concrete to be effective. Small diameter holes, normally no larger than 38 mm (1.5 in.) are spaced over the suspected area. Spacing is typically much tighter than in geomaterials, commonly 1 to 1.2 m, (3 to 4 ft), although initial grouting may be at greater spacing, using typical split spacing to treat suspect areas.

All holes should be wash cleaned of drill cuttings and water pressure tested, Figure 3. Those holes that freely take water or communicate to adjacent holes or the concrete surface, Figure 4, should be indicated for pressure grouting, Figure 5. Holes that do not readily take water, but experience a pressure decay indicating small voids and/or lack of venting of entrapped air, should be identified for vacuum grouting, Figure 6. Prior to grouting, all holes should be blown free of water, Figure 7.

Unlike in rock, even very small voids in concrete can have negative influence on the capacity of a concrete member, if adjacent to the reinforcing steel or in a bearing area. For such work, a stable, cementitious grout with strength similar to the substrate concrete should be used. A pre-blended bagged grout conforming to the Post-Tensioning Institute PTI M55.1-12 Specification for Grouting of Post-Tensioned Structures is a good choice for most applications.

In earlier times, post tensioning tendon ducts were typically filled with unstable cement-water grouts which often resulted in voids in the upper elevations. Because, the purpose of the grout is to protect the enclosed tendons from corrosion, complete filling is necessary, and any voids found, should be grout filled. Because post-tension ducts and the surrounding concrete are usually very tight, they seldom provide for proper venting of entrapped air; filling is thus best accomplished with vacuum grouting.

Grouting of cracks is typically for one of three principal reasons, to stop the flow of water, weld the section back to a monolithic mass, or prevent intrusion of foreign or deleterious elements therein. An essential consideration is

Figure 5. Typical grouting header for circulating injection system. Supply hose on right, return on left. Valve above gauge controls injection rate.

Figure 6. Vacuum grouting header. Valved hose top left is compressed air entering venturi ejector, white hose on right carries away spent air. Upon sufficient vacuum, valves are turned from vacuum to grout.

whether the crack is moving or dormant. A grout that remains somewhat flexible should be used in cracks that are leaking or moving, whereas a high bond material such as epoxy, is ideal where structural bonding is the objective. Where controlling seepage is the objective, chemical solution grout, either based on acrylic or urethane technology is typically used. The material selection depends largely on the nature of the leakage. Acrylic type grouts generally penetrate better and are thus used on fine cracks and seeps, whereas urethane formulations tend to perform better on larger and more active leaks. There are however many different formulations of both generic types, which provide different performance characteristics.

Grouting in masonry

Voids in masonry, especially in very old structures, can be continuous and massive, Figure 10. Exceptional care must be taken when injecting grout, in that large internal forces can be developed as a result of the ever enlarging pool of grout, and unwanted travel and leakage can also occur. I once investigated a project where grout injected on the third floor had run all the way to the base of the wall into a basement area. Leakage is also common through surface defects, which can make the operation quite messy, Figure 11, and result in a marred surface upon completion. Strength of the mortar used in masonry can vary greatly, depending primarily upon age. Prior to about 1940, very low strength lime mortars were common whereas the mortar of modern structures possesses concrete-like strengths. Where cementitious grouts are used, they should match the properties of the original mortar as closely as practicable. Further, one must take care, to not

Figure 7. Blowing hole free of water prior to injection.

Figure 8. Typical epoxy injection from surface. Cracks are pre-sealed and operation can be quite neat.

Figure 9. Urethane injection; defects are intercepted at depth and grout returns to surface can be quite messy.

Figure 10. Large and sometimes continuous voids are common in older masonry.

add more weight to the structure than absolutely necessary, and where large quantities of injected grout are expected, capacity of the foundations to carry the additional load should be confirmed. Very old structures have often undergone previous settlement during their life, and have little capacity for additional loading. Even small additions of dead weight can initiate further movement. In this regard, where large cavities must be filled, expansive polymer foams have been used, in order to minimize the additional load.

Cementitious grouts should not only have strength on the order of the mortar, but also be completely stable (no settlement of solids or bleed) and free of shrinkage. Use of viscosity modifying admixtures to provide thixotropy to the grout will greatly assist in limiting travel and leakage of the grout within the structure. In addition to cement and admixtures, typically used mixes contain up to about four parts of sand, and are usually injected in a thick slurry consistency. To provide stability and shrinkage resistance, the water:cement ratio should be kept as low as practicable and high range water reducing admixtures are advantageous. The exact consistency will depend upon the nature of structure, purpose of the injection, and particulars of the void system, and can vary between a thin fluid to a thick slurry.

Because leakage, which can very much detract from the aesthetics of exposed surfaces is such a problem, it is best to repoint or seal exposed joints prior to any grout injection, Figure 12. This is especially important when resin grouts are used as they can be extremely difficult if not impossible to remove.

Masonry is a composite of different elements, all held together with the mortar. The elements can be of varying size, stiffness, and strength. The

Figure 11. Grout leakage is usually a huge problem when injecting grout into masonry.

Figure 12. It is good practice to repoint and seal masonry surfaces prior to injection.

strength of the mortar bond can vary widely and can be almost zero in older structures. Displacement of the individual elements is thus a continuous risk when grouting masonry, and extreme care must be taken to limit internal pressure and prevent damage of the section being injected.

Closing thoughts

Structural grouting is a much in demand technology and one that many established grouters tend to avoid. This results in substantial opportunity for those that master such work, and provide these services. The procedures have been well established for many decades, so resources are readily available. Likewise grouting materials, both cementitious and resinous are widely marketed especially for structural applications. With such opportunity, every grouter should consider working in this market.

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IN MEMORIAM

Frederick Lionel Peckover 1921-2015

Frederick Lionel (Peck) Peckover passed away on February 8, 2015 in his 93rd year surrounded by his family. Lionel was a pioneer in geotechnical engineering in Canada. He graduated in Civil Engineering from the University of Toronto in 1944 and was encouraged by Dr. Robert Legget to pursue studies in geotechnical engineering which he did through a scholarship at Harvard where he graduated with an S.M. Degree in 1947.

Lionel had a distinguished career in applied geotechnical engineering. He served with The Saint Lawrence Seaway Authority in a senior capacity from 1953 to 1959 where he supervised design and construction of the foundations of locks, bridges and other structures as well as the Seaway channels and dykes. He then joined Canadian National Railways and became Engineer of Geotechnical Services with coast-to-coast responsibility and particular emphasis on such issues as design of railway roadbeds on soft ground, improvement of ballast and reduction of frost heave, and treatment of unstable rock slopes. In 1976 he joined Canac Consulting Group where he carried out terrain appraisal for a proposed high speed rail line from Montreal to Windsor. He retired in 1984. In professional matters, Lionel attended the 1st Canadian Geotechnical Society Conference in Ottawa in 1947 and maintained contact with the Society throughout his career. He published over 40 technical papers and discussions in publications which included The Canadian Geotechnical Journal and the Journal of the American Railway Engineering Association, with contributions also to a number of technical books and manuals. In 1982, in conjunction with the Late Doug Piteau, he received the prestigious Award from the Geological Society of America for the best paper in Engineering Geology.

Jonathan Fannin, Editor

Professor of Civil Engineering, University of British Columbia

Jonathan Fannin

You may recall that, in the GN:December 2014 issue, I sought to compare the origins of current practice for the specification of a geotextile filter with those for the specification of a granular filter. In contrast to a granular filter, the opening size distribution of a geotextile is controlled directly through the process of manufacturing. Accordingly, the properties of a geotextile filter are specified with reference to a characteristic opening size of its fabric, with additional consideration given to the polymer type and also to the strength of the fabric. In the subsequent GN:March 2015 issue, written in collaboration with my good friend and fellow geosynthetics (and rugby!) enthusiast Kelvin Legge, Chief Engineer with the Department of Water and Sanitation in South Africa, we then reviewed select regulatory guidance for granular and geotextile filters, placing emphasis on applications in embankment dam engineering. More specifically, we addressed matters pertaining to base soil-filter layer compatibility because Kelvin, through his involvement with the South African National Committee on Large Dams (SANCOLD), is seeking to update the now wholly-outdated 1985 ICOLD Bulletin 55 on "Geotextile Filters in Dams". In this current article, I have chosen to remain with the theme of filtration applications, and a return to geotextile 'basics'.

Geotextile material properties

Geotextiles are generally made from one of three polymer types: polypropylene (PP), polyester (PET), and polyethylene (PE), with the first two polymers accounting for the majority of geotextile manufacture. The manufacturing process yields three principal styles of geotextile: woven, nonwoven and knitted fabric, with the first two styles accounting for the majority of production and use in filtration applications. The manufacture and use of woven geotextiles in filtration applications pre-dates that of nonwoven geotextiles. The two styles of geotextile are inherently different.

A woven geotextile is made from individual polymer strands that are aligned and interwoven on an industrial loom, yielding a planar fabric. The strand itself is usually a tape, a monofilament, or a multifilament yarn. A fibrillated strand is one that has been intentionally split along portions of its length, as a part of the manufacturing process, to condition its properties.

In contrast, a nonwoven geotextile comprises a layer of many randomly oriented polymer strands that are bonded to obtain a planar fabric. The individual strands are usually a short fibre or a continuous filament. The common methods of bonding are either physical entanglement of the strands, yielding a needle-punched nonwoven geotextile, or thermal fusing of contact points between the strands during a calendaring operation, which produces a heat-bonded nonwoven geotextile.

Inherent differences between each of these manufacturing processes impart differences to the opening size distribution of the fabric and, by association, differences to the capacity for flow of water across the plane of the fabric. Likewise, there is a difference in tensile strength and stiffness that results from the manufacturing process. Accordingly, in

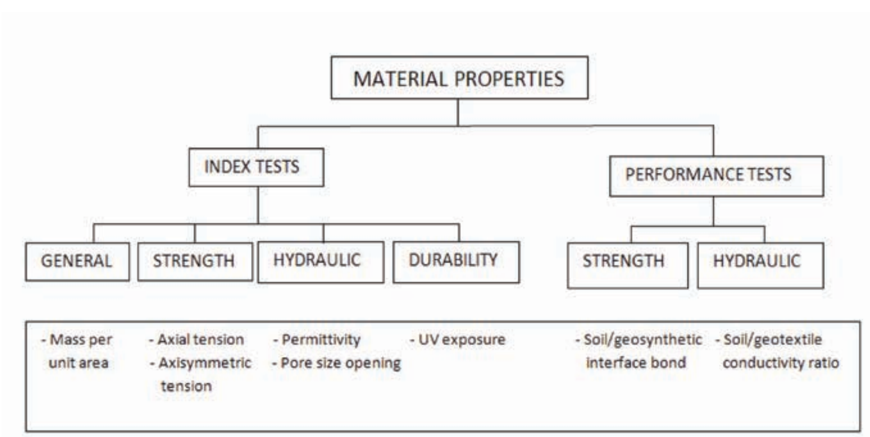

Figure 1. Standard test methods (adapted from Fannin,2001).

design, the assessment of a candidate geotextile for a proposed filtration application must address a series of material properties. A value for each material property is typically obtained from laboratory index tests performed in accordance with the appropriate national or international standard test method. When specifying a geotextile for a filter application, our primary interest lies in determining an index value for the pore size opening, the permittivity and the strength of the fabric (see Fig. 1). Consider now the measurement and reporting of each of these index properties.

Pore size opening of the geotextile

The manufacturing process exerts a significant influence on the pore structure of a geotextile. A woven geotextile exhibits a relatively uniform pore structure that is characterized by openings of nearly constant size and little spatial variability across the surface of the roll. The weave pattern may differ between products, as may the size of the polymer strands used to provide the materials of the warp and the weft, both of which govern the size of the resulting openings between the strands. In contrast, a nonwoven geotextile exhibits a relatively wide range of opening sizes which vary spatially across the surface of the roll. As a result, the pore-size distribution is strongly influenced by factors including the type and density of strands and, in the case of a needle-punched product, the needle-shape, punch density and direction or, in the case of a heat-bonded product, the nature of the contact surface and line-speed of the heated rollers (Bhatia and Smith, 1996). Accordingly, the two main types of geotextile, namely woven and nonwoven, exhibit a significantly different pore structure.

Index test methods used to determine the pore size opening of a geotextile involve (i) variations on reverse sieving of grains through the pores of the fabric (by means of a dry sieving, a wet sieving or a hydrodynamic sieving method, as illustrated in Fig. 2), (ii) the injection or removal of a fluid from the pores of the fabric (by mercury intrusion porosimetry, or a bubble point method, respectively) and (iii) direct image analysis of the pore space. Current design practice is based on methods of reverse-sieving, for which manufacturers report values in their technical literature.

Tabel 1. Variation of pore size opening (µm) with sieving technique (extracted from Van der Sluys and Dierickx, 1990)

Geotextile	O_n (µm)	Dry sieving	Wet sieving	Hydrodynamic sieving
W3	O_{90} O_{98}	278 348	301 387	282 374
W4	O_{90} O_{98}	354 416	307 358	303 360
W6	O_{90} O_{98}	294 339	259 295	225 289
W7	O_{90} O_{98}	253 260	172 210	194 224
NW1	O_{90} O_{98}	179 202	143 195	133 181
NW3	O_{90} O_{98}	204 236	145 191	150 202
NW4	O_{90}	210	189	150
Note: Woven (W) geotextile: Nonwoven (NW) geotextile				

Reverse-sieving methods are suitable for determining the largest pore size openings, which are believed to exert the greatest influence on base soil retention in filtration applications. In dry sieving (see for example, ASTM D4751), different size fractions of glass beads (from small to large, in ascending sequence of size fraction) are sieved through the geotextile by means of a shaking action. The surface of the fabric is pre-treated with an anti-static spray to minimize the influence of attraction resulting from static electricity. In wet sieving (see for example, ISO 12956), a graded mixture of glass beads is sieved by shaking in combination with a continuous spray of water, following pre-treatment of the fabric by a wetting agent. In hydrodynamic sieving (see for example, CAN CGSB 148.1 No.10), a graded mixture of glass beads is sieved by repeated immersion of the geotextile in water and hence alternating flow conditions, without any shaking action.

A common aspect of all three standard test methods (see Fig. 2), is that a gradation analysis of beads passing through the fabric during a standardized duration of shaking or number of immersion cycles is used to infer the size of the largest pore openings in the geotextile. The three methods yield similar but not identical values of opening size (see Table 1). Generally, the dry sieving method yields a relatively larger value of pore size than that obtained from either wet sieving or hydrodynamic sieving (see for example, Faure et al., 1986; Van der Sluys and Dierickx, 1990; Bhatia et al., 1996). Accordingly, when using design criteria for soil retention that relate a characteristic opening size of the geotextile to a characteristic grain size of the base soil, it is important to recognize not only the empirical origin of the design criterion but also the correlation to a particular laboratory test method to determine the opening size of the geotextile.

Hydraulic conductivity of the geotextile

Methods used to determine the permittivity (cross-plane permeability) of a geotextile involve variations on laboratory permeameter testing, the two most common of which are a 'constant-head' method and a 'falling-head' method (see for example, ASTM D4491 and ISO 11058). The geotextile specimen is not subject to any normal load in these index tests, and therefore is tested in an uncompressed state. Where appropriate, provision exists to perform the test under a specified compressive stress (see for example, ASTM D5493 and ISO 10776). Experience suggests the compressive stress yields no significant change to the permittivity of a woven geotextile or a nonwoven heat-bonded geotextile, but causes a reduction in permittivity in nonwoven needle-punched geotextiles. Using image analysis techniques, Palmeira and Gardoni (2000) attribute the reduction in flow capacity to an increase in contacts between the needle-punched fibres, and therefore a greater constriction of pore channels across the plane of the geotextile.

Multiplying the permittivity by the nominal thickness of the geotextile yields a nominal value of permeability or hydraulic conductivity. However, given the range in thickness of different types of geotextile, it is generally recognized that reporting a value of permittivity avoids the potential for any misleading comparison of permeability between different products. As for granular filters, the permittivity values of geotextiles vary over several orders of magnitude.

Geotextile tensile strength

Woven and nonwoven geotextiles exhibit a very different characteristic response to loading which, again, is a direct consequence of the manufacturing process. Woven geotextiles exhibit a significantly greater stiffness upon loading, a response that arises from the preferential alignment of polymer strands during the weaving process. In contrast, nonwoven geotextiles have a random layout of polymer strands that must progressively deform in order to align themselves in the direction of imposed loading. Methods used to determine the strength of a geotextile involve loading it to failure as a result of tensile rupture of the polymer strands (see Fig. 3): the two most common methods are uniaxial-tension testing of a rectangular specimen (using either a full 'wide-width' clamp, else a partial 'grab' clamp) and axisymmetric-tension testing of a circular specimen (using a rod to puncture).

In the 'wide-width' style of test, a rectangular specimen is clamped across its entire width and the load-extension response then measured over a specified gauge length, for loading imposed at a constant rate of axial displacement (see for example, ASTM D4595 and ISO10319). In a variation to this method, only the central portion of a rectangular specimen is clamped over a specified gauge length in the 'grab' style of test (see for example, ASTM 4632). Static puncture resistance is measured by advancing a probe of specified diameter into a specimen that is clamped between circular rings, at a constant rate of displacement, in order to determine the maximum resistance (see for example, ASTM D6241 and ISO 12236). In a variation to this concept, a cone is dropped through a specified distance onto a circular test specimen, in order to measure the

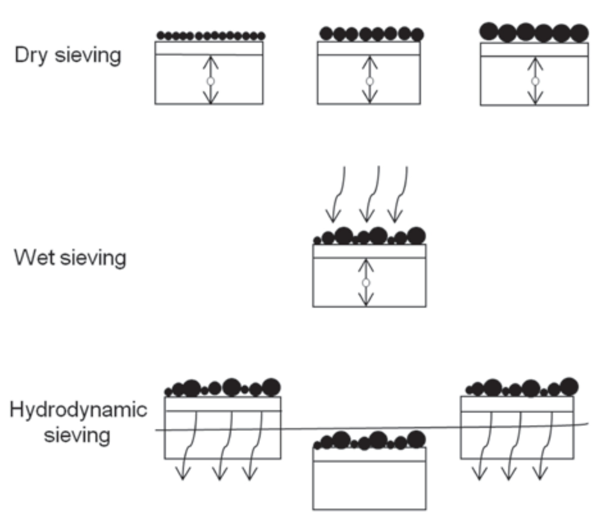

Figure 2. Standard test methods for pore size opening.

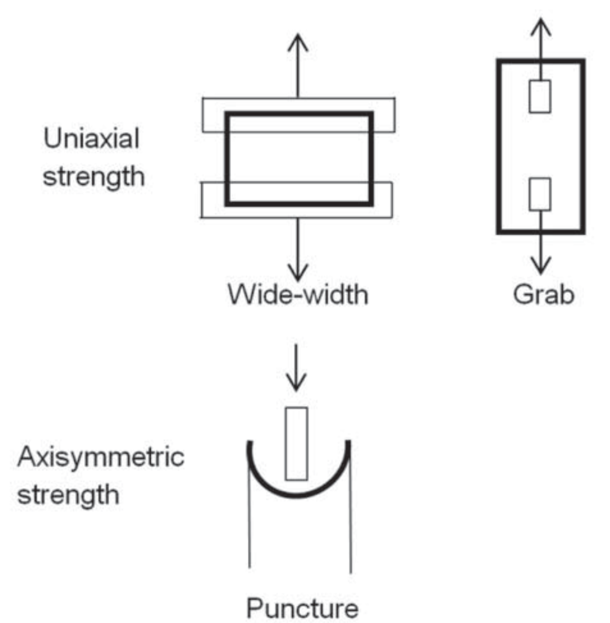

Figure 3. Standard test methods for strength.

extent of penetration that is achieved (BS EN ISO 13433).

Soil-geotextile compatibility

The use of a geotextile in filtration applications is predicated on it having adequate strength to ensure no adverse damage throughout the process of installation (termed 'construction survivability') and that it can also endure, thereafter, the working environment of the application (termed 'durability'). The greatest physical demand on the geotextile is typically encountered during the installation process. The design approach, for construction survivability, requires that the geotextile meet or exceed a required strength category. A qualitative category (for example a low, moderate or high strength requirement) is typically based on the type of field application and anticipated severity of loading imposed during placement and subsequent construction. The qualitative category is then expressed in quantitative form, with reference to a designated range of strength values determined from laboratory index testing .

Thereafter, the geotextile must also be sufficiently durable to ensure it can sustain its intended function over the intended service life of the structure. Durability is evaluated from a consideration of any likely change in the integrity of the geotextile over time, as a consequence of in-service conditions that would cause an unacceptable degradation of its material properties. The general intent is to avoid any degradation that might compromise the ability of the geotextile to act as a filter. Durability varies with the type of polymer and any additives during the manufacturing process, and is governed by physical, chemical and biological influences (see for example, Calhoun, 1972; Koerner et al., 1988; Elias et al., 1999; Elias, 2001; Kay et al., 2004). Accordingly, durability must be evaluated on both a product-specific and a site-specific basis.

Upon selecting a suitably strong and durable geotextile, 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. Therefore, the principal requirements for compatibility are those of (i) soil retention and (ii) cross-plane permeability. They represent competing interests, insomuch as soil retention is assured by smaller pore size openings in the geotextile while, in contrast, a greater cross-plane permeability is associated with relatively larger pore size openings.

Looking ahead to the next article…

In summary, it is widely-accepted practice to select a candidate geotextile for routine construction works with reference to (i) criteria for strength and durability, given the anticipated method of construction service environment, (ii) an empirical rule governing base soil retention, and (iii) an empirical rule governing base soil permeability. The approach has been found conservative, and yields a geotextile filter for which the margin of safety is believed acceptable. However, the exact nature of that margin of safety is not quantified. Accordingly, there is need for a more comprehensive means of evaluating soil-geotextile compatibility in applications that are critical or severe, where filter incompatibility is deemed problematic within the context of either the ultimate limit state (collapse) or the serviceability limit state (deformation). In principle, this would include filter applications where the consequence of failure is believed to be relatively high, else the cost of remedial works is anticipated to be significant. 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 laboratory compatibility testing of a sample of the base soil in combination with that candidate geotextile filter. My next article in this series will address compatibility testing.

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