Volume 29 Number 3 September 2011

GEOTECHNICAL NEWS

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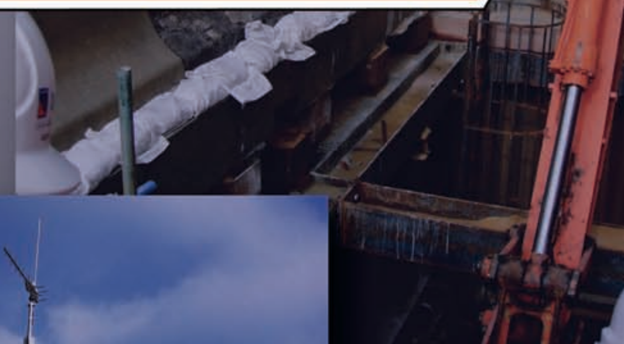

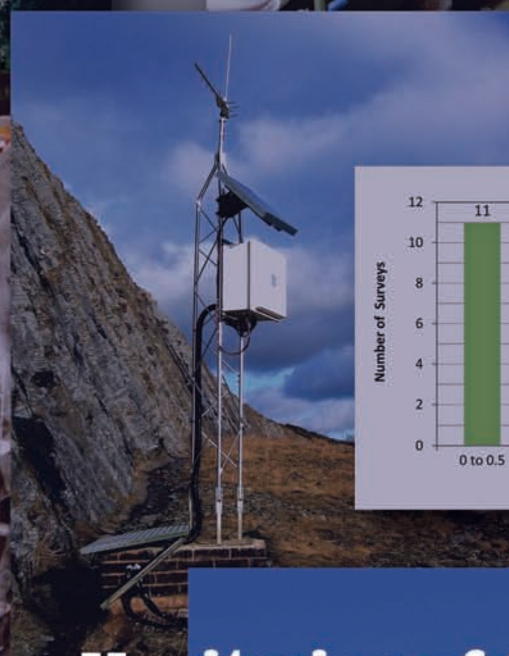

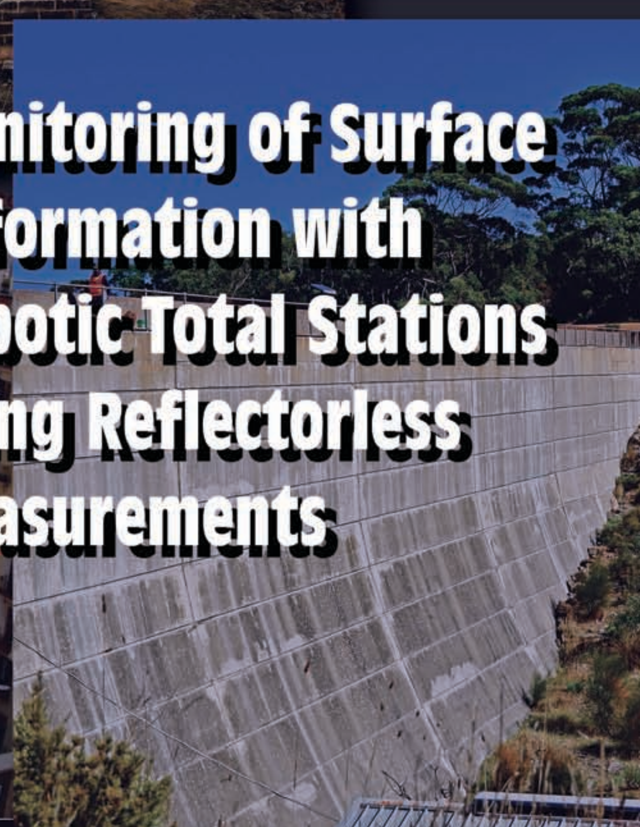

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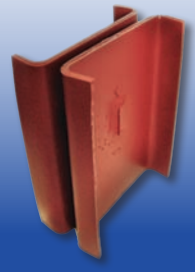

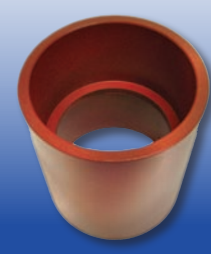

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GEO-INSTITUTE

G-I News

Geo-Institute Governor-Elect Nominee

Patricia J. Culligan, Ph.D., M.ASCE, will assume the position of Board governor for a three-year term beginning in October 2011. Dr. Culligan is a professor of Civil Engineering & Engineering Mechanics and the vice-dean of Academic Affairs of the School of Engineering & Applied Science at Columbia University, New York, NY.

Expand Your Company's Outreach

Need an economical marketing tool to publicize your business? Become a Geo-Institute Organizational Member and receive all the benefits listed below to help grow your business. Download an application at: *http://content.geoinstitute.org/files/pdf/Organizationalbrochure6.16.11.pdf*

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- One free listing as the OM of the month in a monthly G-I eUpdate newsletter.
- Exclusive preference for conference exhibit space at G-I events.
- A link on G-I Web site to your organization's Web site.
- A dedicated page of all Organizational Members in each issue of *Geo-Strata* magazine.

International Events Endorsed by the Geo-Institute

XV European Conference on Soil Mechanics & Geotechnical Engineering
"Geotechnics of Hard Soils - Weak Rocks"
September 12-15, 2011
Athens, Greece
www.athens2011ecsmge.org/

2011 Pan-Am CGS Geotechnical Conference
Geo-Innovation: "Addressing Global Challenges"
October 2-6, 2011
Toronto, Ontario, Canada
www.panam-cgc2011.ca/

International Conference on Ground Improvement and Ground Control
October 30-November 2, 2012
Wollongong, NSW, Australia
www.icgiwollongong.com/

Students

Scholarships for Postgraduate Study in Geotechnical Engineering

ASCE Scholarships and Fellowships
There are numerous ASCE scholarship and fellowship opportunities available for undergraduates and post-graduate students. For information: *www.asce.org/Content.aspx?id=18337*

Eisenhower Transportation Fellowship Program
The Dwight David Eisenhower Transportation Fellowship Program awards fellowships in transportation-related disciplines, including geotechnical engineering. Between 150 and 200 full or partial fellowships are awarded annually. Non-U.S. citizens may apply if they have their Immigration and Naturalization Service granted I-20 or I-551 identification. There are no online applications. The awards program is administered by the Universities and Grants Programs of the Federal Highway Administration (FHWA)'s Office of Professional and Corporate Development.

Student Co-Op and Internship Opportunities

Looking for a co-op or internship opportunity? Then take a look at some of the interesting positions listed on the ASCE website to help you explore your career path. Come back often as new positions are being added all the time.

Co-op opportunities:
http://careers.asce.org/jobs#/results/keywords=coop&resultsPerPage=12&showMoreOptions=true&selectedTab=bti-facets-education/1,false

Internship opportunities:
http://careers.asce.org/jobs#/results/keywords=internships&resultsPerPage=12&showMoreOptions=true&selectedTab=bti-facets-education/1,fals

Members in the News

Gouda Inducted as Maser Consulting Shareholder

Moustafa A. Gouda.

Moustafa A. Gouda., P.E., D.GE, F.ASCE, was recently inducted as a shareholder at Maser Consulting of Red Bank, NJ. Maser is a leading multidiscipline civil engineering company with regional offices in NJ, NY, and PA. Gouda is a principal and director of the Geotechnical / Environmental Services of the firm and has been practicing geotechnical and environmental engineering in the U.S. since 1970. As the director, he is responsible for all geotechnical and environmental engineering work from the preliminary planning phase through consultation during construction of foundations and earth work for structures and the remediation of environmentally-impacted sites. Gouda has been very active in ASCE and the Geo-Institute as a past ASCE Board of Direction member, ASCE treasurer, and a Geo-Institute Board governor.

Mouradian is Geotechnical Engineer of the Year

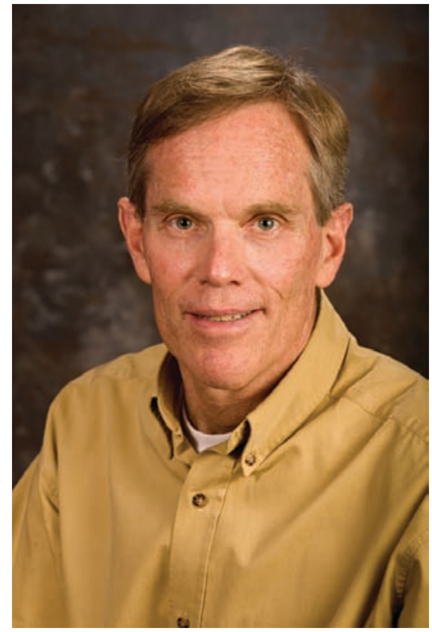
Ara G. Mouradian. P.E., M.ASCE, was cited as the Philadelphia Section of the ASCE 2011 Geotechnical Engineer of the Year. This award was presented to Mouradian at the Section's annual Spring Social and Dinner Dance on May 13, 2011.

Ara G. Mouradian.

Mouradian serves as a senior associate and geotechnical department manager at Gannett Fleming's office in Valley Forge, PA. He has been with Gannett for the past 11years, and serves as vice president of Quantum Geophysics, a division of Gannett. Mouradian has been involved in the geotechnical design of multimillion dollar transit and rail, Department of Transportation, and vital infrastructure projects, both locally and nationally, including local projects such as the Jenkintown Regional Rail station reconstruction, PennDOT's District 6 bridge replacement program, and the Pennsylvania Turnpike-I-95 interchange.

He earned his bachelor's degree in civil engineering at the American University of Beirut in 1984, and a master's degree from Concordia University in Montreal in 1993. He has also completed several doctorate-level courses at McGill University in Montreal and serves as the chair of the Section's Delaware Valley Geo-Institute.

Turner Joins Dan Brown and Associates



John P. Turner.

John P. Turner, Ph.D. P.E., M.ASCE, recently joined Dan Brown and Associates, PC as a senior principal. Turner is Professor Emeritus, University of Wyoming, where he spent the past 25 years teaching and conducting geotechnical engineering research. He has undergraduate degrees in geology and civil engineering and earned his doctorate in geotechnical engineering from Cornell University. He is a co-author of the 2010 FHWA manual "Drilled Shafts: Construction Procedures and LRFD Design Methods" and the author of NCHRP Synthesis 360, "Rock-Socketed Shafts for Highway Structure Foundations", as well as more than 100 technical publications on the topics of deep foundations, earth retention, and landslide stabilization. Early in his career, Turner was an engineering geologist with Herbert and Associates and he maintained his involvement in consulting throughout his academic career. Recent projects include design of rock-socketed drilled shafts for bridges at Pitkins Curve in Big Sur and the Antlers Bridge on I-5 in northern California. He has maintained membership in ASCE for more than 30 years and is a past chairman of the Committee on Deep Foundations. He also is a recipient of the President's Award and the Distinguished Service Award from the ADSC: International Association of Foundation Drilling.

Industry News

A Closer Look at the USACE 2012 Civil Works Budget

The U.S. Army Corps of Engineers' (USACE) non-military funded programs and project budget for 2012 provides an effective pathway for USACE to help create jobs, support economic development and global competitiveness, and restore and protect critical and vital aquatic ecosystems. Unfortunately, as with other federal agencies across the government, this year's budget is less than in prior years. The largest percentage of USACE resources will be used on projects that provide the highest returns on the nation's investment. This includes dam safety projects that are in the greatest need of repair — there are 692 dams that USACE either owns or operates — projects that will reduce the risk of loss of life, projects that will mitigate environmental losses, and advance a number of environmental missions, and on-going projects that they can either complete or make significant progress on. A state-by-state breakdown of the FY 12 Army Civil Works Budge can be found at: *www.usace.army.mil/cecw/pid/pages/cecwm_progdev.aspx*

2011 Tohoku Japan Earthquake Reports

Reports from the 2011 Tohoku Japan Earthquake may be viewed on the Geotechnical Extreme Events Reconnaissance (GEER) website. This material on the website is based upon work supported by the National Science Foundation through the Geotechnical Engineering Program under Grant No. CMMI-0825734 and through the RAPID CMMI-Proposal No. 1034831. For information:

http://geerassociation.org/GEER_Post%20EQ%20Reports/Tohoku_Japan_2011/Cover_Tohoku_2011.html

Editors Note: Watch for the September/October 2011 Geo-Strata magazine whose theme is earthquake geotechnics.

State-of-the-Art Earthquake Shake Table Model on Display

The world's largest earthquake-simulation shake table in Miki City, Japan played a large role in helping scientists design buildings that could withstand large earthquakes. The devastating earthquake in Japan was evidence of the country's preparedness with most of the country's skyscrapers and steel buildings surviving the deadly quake.

Thanks to the efforts of the Japanese National Research Institute for Earthquake Science and Disaster Prevention (NIED), the University of Nevada, Reno now has on display, at the Mathewson-IGT Knowledge Center, a three-dimensional working model of the NIED's Earth- or E-Defense shake table, which simulates the ground motions of an earthquake. They are the first university worldwide to display this model.

Keri Ryan, professor of civil and environmental engineering at the University stated:

"It's a fantastic opportunity. Very few researchers get to do research at the best facilities in the world. Our facility here at the University is the best in the world for bridge testing, and Japan's is the best in the world for testing full-scale buildings." For information: *www.curee.org/EO/miki_table.html*

To submit information for Geo-Strata magazine, or possible posting on the Geo-Institute website at www.geoinstitute.org, send us brief news about your recent honors, awards, special appointments, promotions, etc. High-resolution photos must be sent as separate pdf, tif, or jpeg files. Send to geo-strata@asce.org. Sales-oriented content should be directed to Dianne Vance, Director of Advertising at dvance@asce.org.

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CGS NEWS

Message from the President

Bryan D. Watts, Klohn Crippen Berger Ltd. CGS President, 2011-2012.

My presidential role gets me around to various functions. In May 2011, I attended the Fifth Canadian Conference on Geotechnique and Natural Hazards in Kelowna at the UBC campus there. The first day was a field tour of geologic hazards in the Okanagan valley which is lined with glaciolacustrine silt terraces. After decades of travel to the Okanagan, I finally found out how these glaciolacustrine silts formed in this valley. These silts support extensive residential developments and some of the finest vineyards in Canada. The conference itself included several keynote lectures including a presentation by Mr. Mike Porter of BGC Engineering on “The evolution of geohazard risk management in North Vancouver”. The conference organizers, Dr. Dwayne Tannant, Dr. Rick Guthrie, and their team did an excellent job. Even though I should have known much sooner, I was delighted to learn that Dr. Tannant teaches geotechnical engineering at the University of British Columbia’s Okanagan Campus. The interior of British Columbia has an impressive array of geotechnical issues to attract students.

In June I travelled to Atlanta to attend the ASCE Geo Risk conference. My primary purpose in attending this conference was to make a short lunch time plea to ASCE members to attend our 2011 Pan-Am CGS Geotechnical Conference on Oct. 2 to 6th in Toronto. For those of you reading this article, there is still time to register for Toronto if you want to attend!! On the Sunday before the ASCE Geo Risk conference, I attended a one-day short course on Risk and Reliability of Levees and Dams which featured case histories describing flooding in New Orleans and the vulnerability of the levees on the California delta near Sacramento. The conference featured project case histories, panel discussions, and risk related to the business of geotechnical engineering itself. Even in the southern United States, Canadians were well represented. Two past Presidents of the CGS, Dr. Dennis Becker of Golder Associates and Dr. Suzanne Lacasse of NGI, were there as were Mr. Roger Jinks, past President of AMEC Earth & Environmental and Dr. Fenton of Dalhousie University. Dr. Fenton was one of the conference organizers. I met Dr. Paul Mayne of Georgia Tech for the first time and was surprised to learn that he was born in Newfoundland. Canada seems to grow and export hockey players, hard rock miners, and geotechnical engineers!!

As I write this message, it has been announced that two of our CGS members have been inducted into the Canadian Academy of Engineering. These members are Dr. Dennis Becker of Golder Associates and Dr. David Sego of the University of Alberta. "The Canadian Academy of Engineering (CAE) is the national institution through which Canada's most distinguished and experienced engineers provide strategic advice on matters of critical importance to Canada." This recognition of the contribution to engineering of both of these CGS members is most appropriate. We congratulate them both.

There seem to be unlimited opportunities for young geotechnical engineers to learn and prosper at the present time. It has not always been so. When I first graduated in 1974, there was almost no work for geotechnical engineers in British Columbia so off went most of my class to Alberta. Then came the early 1980s when geotechnical unemployment was at an all time high, at least in Western Canada. Thereafter employment prospects grew with some ups and downs and, now, we appear to be in a golden period where geotechnical engineers are in great demand, not only in Canada but around the world. Canadian universities are graduating more engineers who choose geotechnical engineering as a career more than ever. Yet this buoyancy in our profession is not reflected in the level of membership of the CGS. Why is this so?

We know that one of the reasons our membership has levelled off is because so many stalwart members are retiring. The first few generations of geotechnical engineers are now passing through the ranks. The notable exception is Gordon McRostie who has attended every annual CGS Conference, save one! An essentially level membership may reflect that the originators of our Society have deeper ties than those who follow them. Or it may be that our society is not as relevant to the current generation as it once was. There is little doubt that the present generation of geotechnical engineers have inherited a technical society that was built by dedicated individuals and continues to prosper.

Our VP, Finance, Mr. Peter Gaffran, tells me that for every $200 in membership dues we pay out over $300. The differential comes from conference profits and sales of the Canadian Foundation Engineering Manual. So, our members get good value for their dues.

So, what are the issues with membership? The CGS, under the guidance of our Past-President, Michel Aubertin, have struck a Membership Task Force Committee led by Dr. Richard Bathurst, as Chair. This Committee will try to identify incentives for young engineers to join the CGS. In the meantime, please encourage younger geotechnical engineers to join the CGS which can be an essential part of their technical and professional development. Participation in the CGS is one window into the national culture of engineering which is vital to the continued prosperity of the country. With hard work our young geotechnical engineers can achieve what CGS members, Drs. Becker and Sego, have just achieved; a voice at the very highest levels of engineering in our country.

Le Message du Président

Mon rôle de Président m'amène à avoir plusieurs fonctions. En mai 2011, j'ai participé à la cinquième Conférence canadienne sur la géotechnique et les risques naturels qui se tenait sur le campus de l'université de Colombie Britannique à Kelowna. La première journée consistait en une visite de terrain associée aux risques géologiques dans la vallée de l'Okanagan, qui repose sur des terrasses de silt glaciolacustres. Après des dizaines d'années à parcourir l'Okanagan, j'ai finalement compris comment ces silts glaciolacustres s'y sont déposés. Sur ces silts repose un imposant développement résidentiel ainsi que quelques-uns des meilleurs vignobles du Canada. La conférence elle-même incluait plusieurs présentations vedettes, dont celle donnée par le Dr Mike Porter de BGC Engineering sur «L'évolution de la gestion des risques associés à la géologie de Vancouver Nord». Les organisateurs de la conférence, Dr Dwayne Tannant, Dr Rick Guthrie et leur équipe ont fait un excellent travail. Bien que j'aurais dû le savoir bien avant, j'ai été ravi d'apprendre que le Dr Tannant enseigne la géotechnique au campus de l'université de Colombie Britannique à Kelowna. Cette région centre de la Colombie Britannique regorge de contextes géotechniques susceptibles d'attirer des étudiants.

En juin, je me suis rendu à Atlanta afin d'assister à la Conférence ASCE Geo Risk de la Société américaine de génie civil. Le but premier de ma présence à cette conférence était d'inciter, lors d'une courte pause lunch, les membres participants à assister à notre Conférence Pan-Am SCG 2011sur la géotechnique qui se tiendra à Toronto du 2 au 6 octobre prochain. D'ailleurs, si vous lisez ce texte et que vous avez l'intention d'assister à la conférence de Toronto, sachez qu'il est encore temps de s'inscrire! Le dimanche précédent la Conférence ASCE Geo Risk, j'ai participé à un cours d'une journée sur les risques et la fiabilité des barrages et des digues dans le contexte des inondations qu'a connues la Nouvelle-Orléans et sur la vulnérabilité des digues du delta de Californie près de Sacramento. La conférence proposait des cas d'étude, des sessions plénières ainsi que des échanges portant sur les risques associés au volet affaires de la géotechnique. Même si la conférence se déroulait dans le sud des États-Unis, les Canadiens étaient bien représentés. Deux anciens présidents de la Société canadienne de géotechnique; Dr Denis Becker de Golder Associates et Dr Suzanne Lacasse de l'Institut norvégien de géotechnique étaient présents, de même que M Roger Jinks, ancien président de Amec E&E et Dr Fenton de l'université de Dalhousie. D'ailleurs, le Dr Fenton était un des organisateurs de la conférence. J'ai eu l'occasion de rencontrer Dr Paul Mayne de Georgia Tech pour la première fois et j'ai été surpris d'apprendre qu'il était né à Terre-Neuve. Le Canada semble donc produire et exporter des joueurs de hockey, des mineurs de roche dure ainsi que des ingénieurs en géotechnique!

Alors que je rédige ce message, l'annonce de l'intronisation de deux membres de la SCG à l'Académie canadienne d'ingénierie vient d'être faite. Ces deux membres sont Dr Denis Becker de Golder Associates et Dr David Sego de l'université d'Alberta. «L'Académie canadienne du génie(ACG) est l'organisme national par l'entremise duquel les ingénieurs les plus chevronnés et expérimentés du Canada offrent au pays des conseils stratégiques sur les enjeux d'importance primordiale». La reconnaissance de la contribution à l'ingénierie de ces deux membres de la SCG est très grandement justifiée. Nous les félicitons tous les deux.

Des opportunités illimitées d'apprentissage et de prospérité semblent présentement s'offrir aux jeunes ingénieurs en géotechnique. Cela n'a pas toujours été le cas. Quand j'ai gradué en 1974, il n'y avait presque pas de travail pour un ingénieur en géotechnique en Colombie-Britannique, ce qui a conduit la majorité des membres de ma classe à partir pour l'Alberta. Et puis, il y a eu le début des années 80, avec son creux sans précédent au niveau de l'emploi en géotechnique, du moins dans l'Ouest canadien. Bien que les perspectives d'emploi aient connu depuis une croissance en dents de scie, nous semblons connaître présentement une période dorée, où les ingénieurs en géotechnique sont en grande demande, non seulement au Canada, mais aussi à travers le monde. Les universités canadiennes forment plus que jamais des ingénieurs qui choisissent la géotechnique comme domaine de carrière. Cette montée de la profession ne se reflète cependant pas dans les adhésions à la SCG. Pourquoi est-ce ainsi?

Nous savons qu'une des raisons du plafonnement des adhésions est que bon nombre de fidèles partisans de la première heure prennent leur retraite. Les premières générations d'ingénieurs en géotechnique font place aux suivantes. Une exception digne de mention est Gordon McRostie qui a participé à toutes les conférences annuelles de la SCG, à l'exception d'une! La stagnation des adhésions peut également refléter le fait que les membres fondateurs de la Société ont montré un plus grand sentiment d'appartenance que ceux qui les ont suivis. Ou peut-être est-ce que la Société n'apparaît plus aussi pertinente à la génération présente qu'elle ne l'a été pour les générations antérieures. Il ne fait aucun doute que la génération présente d'ingénieurs en géotechnique a hérité d'une société savante qui a été bâtie par des individus particulièrement dévoués et qui continue à prospérer.

Notre Vice-président aux finances, M Peter Gaffran, me mentionnait que pour chaque 200$ de frais d'adhésion versé par les membres, la Société en débourse 300$. La différence est comblée par les profits générés par les conférences annuelles ainsi que par la vente du Manuel canadien d'ingénierie des fondations.

Alors, comment aborder la question des adhésions? La SCG a constitué un comité spécial, guidé par un ancien Président, Dr Michel Aubertin et qui a pour président Dr Richard Bathurst. Ce comité essaiera d'identifier des incitatifs les jeunes ingénieurs en géotechnique à adhérer à la SCG. Entre-temps, s'il vous plaît, encouragez les jeunes ingénieurs en géotechnique à joindre les rangs de la SCG en leur faisant valoir que la Société prendra part à leur développement technique et professionnel. La participation à la SCG est une fenêtre à la culture de l'ingénierie qui est essentielle à la poursuite de la prospérité du pays. Par leur travail acharné, nos jeunes ingénieurs en géotechnique pourront réaliser ce que nos membres de la SCG, Dr Becker et Dr Sego et bien d'autres avant eux ont réalisé, c'est-à-dire obtenir une voix aux niveaux les plus élevés de l'ingénierie dans notre pays.

From the Society

Upcoming Conferences

65th Canadian Geotechnical Conference
September 30 - October 3, 2012
Call for Abstracts

The Canadian Geotechnical Society (CGS) and the Manitoba Section of the Canadian Geotechnical Society invite you to the 65th Canadian Geotechnical Conference. The Conference will be held at the Fairmont Hotel located in downtown Winnipeg, Manitoba, Canada from **September 30 October 3, 2012.** The "GeoManitoba 2012 Building On The Past" conference reflects the heritage of geotechnical engineering in Canada and how our past will help us going forward in new research, developments and advancements in geotechnical engineering. It also reflects the ever increasing need to restore or upgrade our country's aging infrastructure. The official languages for the conference will be English and French.

Described as the "cultural cradle of the nation" by one of Canada's national newspapers, Winnipeg has a long tradition of developing its arts community, supporting countless galleries, museums, theatres, dance companies and music organizations. Winnipeg also has one of the highest number of restaurants per capita of any city in North America. The Fairmont hotel is located within walking distance of the historic Exchange district and the Forks Market, along with several museums and galleries. Winnipeg's downtown has been experiencing a rejuvenation in recent years with construction of MTS Centre (the home of our newly returned Winnipeg Jets), The Museum for Human Rights which is presently under construction, and Manitoba Hydro Place (which has won several international awards for its innovative design). Please join us to enjoy Winnipeg's rich culture and experience friendly Manitoba hospitality firsthand!

The organizing committee of the conference invites members of the Canadian and International communities to contribute recent research developments and advancements of geotechnical engineering, cold regions engineering, geo-environmental engineering and hydrogeology. The conference will cover a wide range of topics, including special sessions that are of local and national relevance to the fields of geo-engineering. In addition to the technical program and plenary sessions, the conference will include a complement of local tours, workshops and short courses.

Authors are invited to submit abstracts of a maximum **400 words** through the conference web site (www.CGS2012.ca) which will be launched just prior to the 2011 Pan-AM CGS conference in Toronto. The abstract can be written either in English or French. **The deadline for abstract submission is January 27, 2012**. Invitations for submission of full papers will be sent to authors whose abstracts are accepted by the conference's Technical Committee by **February 27, 2012.** The submitted papers will be reviewed prior to final acceptance for inclusion in the conference proceedings, which will be also available on CD-ROM. At least one author of an accepted paper must register for the conference.

Abstracts should generally fall within the following topics, but sessions will be added for groups of abstracts with common themes not listed below:

Case studies, case histories and papers related to revitalization of aging infrastructure are actively solicited. Papers featuring innovative analysis techniques and solutions, as well as research (recent and/or future trends), are strongly encouraged.

Fundamentals

- Engineering geology
- Foundation Engineering
- Geoenvironmental
- Landslides / Slope Stability / Slope Engineering
- Reliability-Based / Limit States Design
- Risk Assessment
- Rock Mechanics
- Soil Mechanics
- Seepage / Groundwater
- Cold Regions Geotechnology
- Soil Stabilization

Geotechnical

- Revitalization of Aging Infrastructure
- Reliability-based / limit states foundation design
- Geohazards
- Retaining walls / MSE walls
- Brownfields and Redevelopment
- Mine Site Remediation
- Design of Earth Dams
- Design of Clay Liners
- Marine Geotechniques
- Non-textbook Soils/Waste Soils
- Harbour and Shoreline Geotechniques

Hydrogeology

- Aquifer Sustainability
- Mine Waters
- Source Water Protection
- Coastal Aquifers
- Paleogroundwaters
- Water Supply Protection
- GUDI Assessment and Protection

Cross-Disciplinary

- Geoenvironmental Sustainability
- Instrumentation

Questions regarding sessions, topics and technical program should be directed to the Technical Committee contacts given below:

For General Inquiries

Gil Robinson
Dyregrov Robinson Inc.
Conference Chair
email: gilrobinson@mymts.net

For Technical Questions

Kent Bannister
Manitoba Hydro
Program Chair
email: kbannister@hydro.mb.ca

14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering and 64th Canadian Geotechnical Conference

The Canadian Geotechnical Society and the International Society for Soil Mechanics and Geotechnical Engineering invite you to the 14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering (PCSMGE), the 64th Canadian Geotechnical Conference (CGC) and the 5th Pan-American Conference on Teaching and Learning of Geotechnical Engineering (PCTLGE) at the Sheraton Centre Hotel in Toronto, Ontario, Canada from October 2 to 6, 2011. Details for the conference are located on the website, www.panam-cgc2011.ca.

11th International Symposium on Landslides (ISL) and the 2nd North American Symposium on Landslides (NASL)

Join us in Banff, Canada in June 2012 for ISL/NASL 2012

The Canadian Geotechnical Society, the Association of Environmental and Engineering Geologists and the Joint Technical Committee on Landslides (JTC-1) invite you to the 11th International Symposium on Landslides (ISL) and the 2nd North American Symposium on Landslides (NASL) at the Fairmont Banff Springs Hotel in

Banff, Alberta, Canada from June 3 to 8, 2012. The theme of the symposium will be Landslides and Engineered Slopes: Protecting Society through Improved Understanding.

The Technical Committee invites participants from industry, government and academia to submit abstracts pertaining to the investigation, classification, monitoring, analysis and mitigation of landslides. Case studies together with papers featuring innovative analysis techniques and solutions, as well as research (recent and/ or future trends), are strongly encouraged.

Canadian Foundation for Geotechnique

AMEC and Stantec are Honoured by the Canadian Foundation for Geotechnique

Mr. Doug VanDine, President of the Canadian Foundation for Geotechnique, is pleased to announce that **AMEC Environment & Infrastructure** and **Stantec** are the first two companies to be honoured by the Foundation as *Legacy Corporate Sponsors*. The *Legacy Corporate Sponsor Program,* established by the Foundation in early 2011, is similar to its *Legacy Donor Program* for individuals that was established in 2008.

The *Legacy Donor Program* recognizes and honours **individuals** who donate, cumulatively at least $25,000 to the Foundation in the form of cash, securities or bequests. Donations can be made by an individual or by a group of individuals to honour a colleague. Contributions can be targeted to one of the Foundation's existing initiatives or to the Foundation for unspecified purposes.

The *Legacy Corporate Sponsor Program* recognizes and honours **corporations** that donate, cumulatively at least $30,000 to the Foundation. Contributions are typically made to help sponsor Cross Canada Lecture Tours, but can also be targeted to one of the Foundation's existing initiatives or to the Foundation for unspecified purposes.

With both programs, the amount of the donations is never disclosed, and the parties are honoured at the Canadian Geotechnical Conferences. Each year, when the Foundation honours its new *Legacy Donor Sponsors* and *Legacy Corporate Sponsors*, all previous honourees are be recounted. At the 2011 Pan-Am CGS Geotechnical Conference in Toronto (October 2 to 6) **AMEC Environment & Infrastructure** and **Stantec** will be honoured.

AMEC Environment & Infrastructure, until June 2011 known as AMEC Earth & Environmental, has been a generous sponsor of the Foundation and the CGS Cross Canada Lecture Tours since 2001. AMEC provides engineering and project management services to the world's oil and gas, minerals and metals, clean energy, water and environmental sectors. The company employs more than 25,000 people in around 40 countries worldwide.

When notified of being honoured by the Foundation, Brian Ross, P.Eng., Executive Vice-President of AMEC Environment & Infrastructure operations in Western Canada and South America, expressed his thanks and went on to say *"AMEC's team of professionals are working on a wide range of projects involving environmental, infrastructure planning and design, and construction related services around the globe. Geotechnical engineering plays a very important role in many of these projects. We have a culture of excellence and our people are our biggest assets. AMEC Environment & Infrastructure supports the Foundation's mission of recognizing and fostering excellence in the geotechnical field in Canada."*

Starting in 2000, **Jacques Whitford** began a continuing relationship with the Foundation by sponsoring the CGS Cross Canada Lecture Tours. In 2009 Jacques Whitford amalgamated with **Stantec**. Stantec's geotechnical services are focused on infrastructure, including dams and levees, transportation facilities, abandoned mine lands, landfills, waterfront engineering and electric utilities, and geotechnical program management for its national and international clients.

Michael Whitford, P.Eng., co-founder of Jacques Whitford and Vice President Vice President Geotechnical Canada of Stantec expressed his thoughts of being honoured by the Foundation this way. *"Stantec, and our legacy company Jacques Whitford, support the Canadian Foundation for Geotechnique because we believe that excellence in geotechnique should be recognized and supported, regardless of whether that excellence is in the public sector, the private sector, or academia. Geotechnique was the original tenet of our business and today remains the basis of one of our core lines of practice. The Foundation's objectives mirror our own internal recognition programs. Striving for excellence and continuing education keep our industry dynamic, and we are proud to be a supporter of the Foundation's awards programs and the Cross Canada Lecture Tour series"*.

The Foundation expresses its sincere thanks for all the financial support is receives from individuals, local CGS sections and corporations. For more information on the Foundation, check out its new bilingual website www.cfg-fcg.ca.

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Geotechnical Instrumentation NEWS

Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the sixty-seventh episode of GIN. Two articles this time.

Interchangeability of Inclinometer Probes

The first article, by Brian Tigani and Rolando Rongo of Monir Precision Monitoring Inc., Ontario, Canada, provides useful practical guidance on monitoring with MEMS digital inclinometer probes. I welcome such nuts-and-boltsy help from experienced users. Any more out there?

Monitoring of Surface Deformation with Robotic Total Stations Using Reflectorless Measurements

The second article, by Damien Tamagnan and Martin Beth of SolData Group in Spain and France tells us about a recent development whereby measurements of vertical deformation can be made by robotic total stations without the need for prisms. This allows us to monitor ground surfaces such as road pavements without obstacles on the surface and consequent interruption to traffic.

Yes, I know that I don't normally publish articles that are written by authors with a commercial interest in the subject, in an effort to keep GIN as a totally professional source of information. But I decided that this article added enough to our toolbox so that I'd make an exception.

Confusion about Initial Readings and Baseline Readings

Some years ago I participated in writing a guide instrumentation specification for a major construction project. Funding regulations mandated that various tasks, including reading the instruments, had to be included in the general contractor's scope of work. In the specification for reading instruments I adopted the term "formal initial readings" (FIRs). These were intended as readings to which all subsequent readings would be referred, hence indicating changes. The FIRs needed to be taken after all installation effects had disappeared, such as 'settling down' after drilling, grouting, welding etc., (remember that swelling of bentonite can cause either reduction of pore water pressure by drawing water out of the pores or increase of pore water pressure by pressing on the soil, and that equilibrium may not be reached for a while), and therefore wording was included to specify timing with respect to installation. FIRs also needed to take into account any non-repeatability from reading to reading, and therefore wording was included to specify how many individual readings were required and how to use these to create an FIR.

This wording has been copied for other construction projects. I've recently learned that others may not fully appreciate the logic behind FIRs, and are confusing them with baseline readings. So I'll try to define what baseline readings are and why they are entirely different from FIRs.

Baseline readings are readings taken over a period of time, before any construction starts, to help in the definition of changes that occur from causes other than construction. For example, seasonal changes in groundwater levels often cause deformation of structures. Tidal and moisture content changes can do the same thing. Climatic changes such as temperature and incidence of sunlight can cause substantial deformation of structures. If these naturally occurring changes are not documented, the task of evaluating measured changes is severely hampered, and it requires significant engineering judgment to adjust day-to-day measured changes to discount those that have nothing to do with construction.

In summary, formal initial readings and baseline readings are entirely different things, and formal initial readings come first.

On a Related Subject

I'm working with a colleague to put together answers to the question, "How should we determine response values (RVs, a.k.a. trigger levels and hazard warning levels)?", and hope to include this in a later GIN. A few thoughts now:

- Don't ignore changes during the green RV period by simply waiting for the green flag to change to amber. Trends during the green period can give useful forewarning.
- Early RVs can be based on calculated changes, whereas later RVs can be based on (unrelated) tolerable changes.

- RVs must recognize the changes that occur from causes other than construction.
- RVs should be several times larger than the accuracy of measured changes (those last four words are very carefully chosen).

Closure

Please send contributions to this column, or an abstract of an article for GIN, to me as an e-mail attachment in MSWord, to john@dunnicliff.eclipse.co.uk, or by mail: Little Leat, Whisselwell, Bovey Tracey, Devon TQ13 9LA, England. Tel. +44-1626-832919.

Alla salute! (Italy)

P.S. For those of you who are not long term readers of GIN, here's the background to the line just above. Soon after GIN was born in 1994 a colleague gave me a beer mat inscribed with about a dozen drinking toasts, in different languages. We agreed that they would make appropriate endings to GIN 'columns'. "Alla salute!" is the sixty-seventh different toast to end a column.

Alla salute!

Interchangeability of MEMS Digital Inclinometer Probes

Brian Tigani and Rolando Rongo

This article examines the data collected with Micro-Electro-Mechanical Systems (MEMS) inclinometer probes, using inclinometer probes manufactured by RST Instruments Ltd.

History

Inclinometer systems consist of casings with alignment grooves, inclination sensing probes, communication cables and readout devices. The casing is placed into the ground or attached to a structure which is anticipated to move and the equipment is used to monitor any deformation perpendicular to the alignment of the casing.

Stanley D. Wilson, creator of the "slope inclinometer" in 1954 and co-founder of Slope Indicator Company produced the first production model inclinometer in 1957. Wilson originally attached his inclinometer casing to sheet piling. There has been a tendency to use inclinometers more for dam and soil shear measurements. The majority of inclinometers at Monir Precision Monitoring Inc. are used for monitoring support of excavation walls.

The Survey Process

Analogue vs. Digital (MEMS)

Analogue inclinometer probes have been in use since 1957; however they are not interchangeable. Each probe has its own characteristics and is sensitive to shock and temperature (range: -20 to +50 deg. C) which amplify these characteristics. As a result, the probe used to make an initial reading was thereafter the only probe which could be used reliably to survey that installation. Unlike analogue systems, the MEMS are less sensitive to shock and temperature (range: -40 to 70 deg. C), minimizing such probe characteristics. Also the MEMS system which was tested aids in technician repeatability. For example, the cable grip ensures all technicians read at the same top reference mark, unlike the pulley/cleat assembly typically used with analogue systems.

Data Gathering and Analysis

If different probes survey installations differently, data gathering with only one probe may be a liability in the event of later unavailability for reasons such as; damage, loss, calibration or scheduling conflicts. To address this concern, Monir chooses to take initial readings of every installation with two probes; in the past with analogue and presently with digital. This ensures accurate surveys could always be collected. If a probe is away for its yearly calibration or simply not available, a survey can then be taken without delay.

When first introduced at Monir, we employed the same protocols with the MEMS system, as it was understood that these probes were also not interchangeable. The manufacturer states data gathered from *one* probe are repeatable over 25m of depth to within 2mm, (RST manual, October 12, 2010).

When we make initial surveys of an installation, multiple sets of surveys are taken using two probes to confirm the casing initial position within 1mm over 25m of depth (as compared with 2mm for the manufacturer's specifications). This practice was adopted when attached (not borehole) installation depths in the Toronto area were short, typically 15m. Installation depths for this study ranged from 6.7m to 32.3m.

As we gathered data using different MEMS equipment we began to see a clear trend of interchangeability based on our above criterion. With this trend we questioned the duplicated survey approach and decided in September of 2008 to further analyze our data. It was one thing to get repeatable initial surveys but another to ensure such repeatability for moving installations.

The only way to show that probes were interchangeable was to take consecutive surveys with *multiple* systems and use our above criterion for repeatability. So in addition to two sets of initial readings with different probes, we

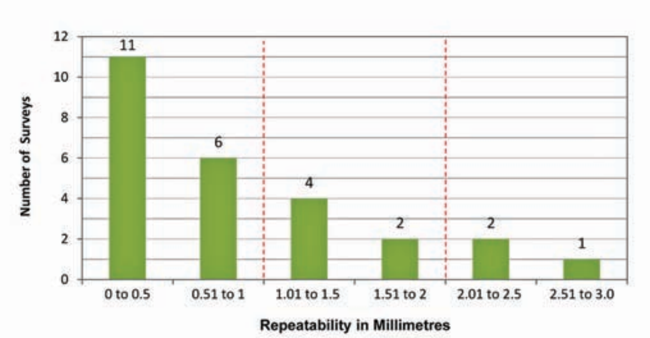

Figure 1. Survey repeatabilities with multiple probes for borehole installations.

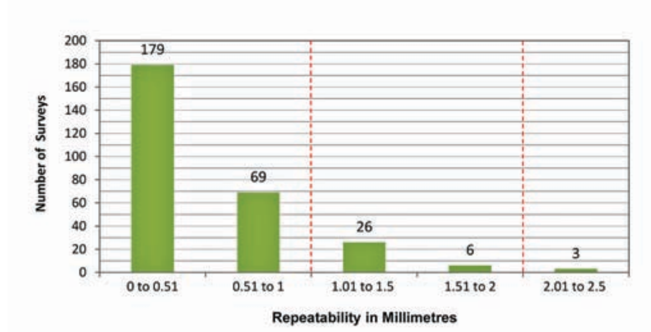

Figure 2. Survey repeatabilities with multiple probes for attached installations.

surveyed using a second probe throughout construction projects. These data were used to build our database.

Study Specifics and Results

Consecutive surveys (A0/A180 data only) were taken with only one probe at two types of installation:

- Borehole installations use ABS casing backfilled with grout into a hole in a suspected zone of ground movement.
- Attached installations use ABS casing attached to piles or rigid structures and backfilled with grout into a caisson wall.

The readings collected were compared and all found to be repeatable to both our criteria and the manufacturer's specifications.

Further consecutive surveys again (A0/A180 data only) using as few as two probes or as many as four probes, were compared for repeatability, with the flowing results:

- Borehole installations. Figure 1 shows 26 surveys. As can been seen, 17 were repeatable within our criterion and 23 met the manufacturer's specifications, representing 65% and 88% respectively.
- Attached installations. Figure 2 shows 283 surveys. As can be seen, 248 were repeatable within our criterion and 280 met the manufacturer's specifications, representing 88% and 99% respectively.

Conclusions

Borehole installations represent 10% of Monir's inclinometers. Typical borehole installations are more out of plumb, have more undulations and undergo more movement than attached installations, and we believe that this is the reason for the poorer repeatability.

As attached installations on piles for excavation support are the majority of Monir's installations, we plan on continuing to focus our attention on these.

Based on the results of this study, Monir will consider probes to be interchangeable for attached installations to the manufacturer's specifications. We will however strive to implement procedural improvements which will achieve the same repeatability for our criterion.

Brian Tigani and Rolando Rongo
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Monitoring of Surface Deformation with Robotic Total Stations Using Reflectorless Measurements

Damien Tamagnan and Martin Beth

Introduction

Real time monitoring using Robotic Total Stations (RTS) over tunnel excavations in the proximity of diaphragm walls or other construction generally includes monitoring of buildings and ground movements. The challenge in the case of roads and pavements is to leave the site free of any obstacles and to observe surfaces automatically in order to respond to real time monitoring criteria, without installing sensors. The aim is to avoid problems caused by the interruption of traffic and above all, for safety reasons, the danger of making traditional manual topography measurements on an active road.

New generations of robotic total stations allow Reflectorless Surface Point

(RSP) measurements, thanks to a laser beam aimed directly on the surface.

Nowadays two methods of computing surface settlement exist:

- The standard method (single points directly measured by the total station)
- The mesh method (treatment of a number of points to geographically smooth the results).

We have used both methods extensively in Europe over the past few years. Both have advantages and drawbacks.

This article presents the generalities of the technique, its potential limitations and requirements, and then briefly presents two sites where both standard and mesh methods were used.

How does it Work?

3D monitoring with a RTS consists of a zero measurement of a network of points measured in three fixed directions to be able to follow this network over time. Preferably the baseline measurement is performed previous to any construction work. A record of the weather conditions (temperature, pressure and humidity) and all the factors that could influence the measurements is very important.

An automatic 3D monitoring system able to measure surface deformation 24 hours a day is made up of a total station equipped with a reflectorless distance meter and a personal computer which can be operated remotely with specific software able to drive the total station to predetermined locations of the points that are to be monitored. We will refer to this entire system as Reflectorless Robotic Total Station (RRTS) for the rest of this article[1].

[1] *The commercial name of the whole system as developed and used by Soldata is "CENTAURE". This name now appears regularly in articles and specifications, but for the rest of this article and for future generic use we suggest the use of the term RRTS for Reflectorless Robotic Total Stations. As for RTS, the term RRTS will apply both to the total station being used and to the whole system, including all software and data treatment processes.*

Figure 1. Example of Reflectorless Robotic Total Station (RRTS) installation able to measure RSPs and prisms.

During each monitoring cycle the instrument sights at two or three groups of points (see Figure 1):

- RSPs on a flat, homogeneous and planar surface for which vertical deformation is to be monitored. RSPs are not physically marked and are not physical objects: They are just a location on the ground at which the RTS is sighting.
- The stable reference prisms, which permit computation of the correct position and the orientation of the total station.
- If necessary, the same total station and software can sight monitoring prisms installed on structures to be monitored in 3D, the same as for a standard RTS

On completion of the cycle (typically 20-40 minutes, depending of the number of points), the raw data are sent to the database via Wi-Fi or 3G.

If both the availability and the distribution of the values meet the quality criteria then the height of the RSP is calculated and can be published in real time via a web-based GIS. Treatments include sliding statistical analyses of the data. These methods allow removal of any accidental errors produced by the total station, and greatly improve the precision of the data

This system can also trigger alarms sent by SMS or e-mail if predetermined thresholds are exceeded.

System Limitations

References

RRTSs are nearly always installed inside the area of influence of the work site where settlements are expected. The position of the total station and the associated prisms are computed based on reference prisms located outside the area

The adequacy of the whole system is based on the quality of the reference prisms. They need to be:

- Well distributed to guarantee the robustness of the system
- Located in a stable zone outside the area of influence
- Located at a distance which depends on the precision required

Range

Depending on the type of total station used, the range of the distance meter is limited (typically 60-70m). To guarantee a good reflection quality of the laser beam the angle of incidence on the measured surface is also a criterion that influences the range of the measured RSP. Finally the surface

Figure 2. Total stations in Amsterdam sighting prisms and RSP. In this case two total stations are installed to allow a larger number of points to be measured more often. Both total stations can measure both RSPs and prisms.

characteristics (colour, smoothness, material) also affect the range and the precision. All these elements shall be taken into account when designing a site setup.

Obstructions

Due to their location in roads and pavements the RSPs are likely to be randomly hidden by obstacles such as pedestrians or cars, in which case the total station will take the measurements but the data will be filtered during the acquisition chain.

Weather Conditions

Rain, snow and fog clearly downgrade the emitted distance meter signal and can prevent some of the measurements from being made. Snow, leaves or mud on the ground will also change the height of the apparent RSP.

Results of Field Studies, Standard and Mesh Methods

In this section we will present both methods, their advantages and their drawbacks and an assessment of the precision.

The Standard Method

For the standard method the RRTS is simply programmed to sight the road surface in predefined horizontal and vertical angles. The RRTS measures the inclined distance, and the software calculates the variations in vertical position (only) of the point.

It is possible to automatically estimate an adjustment of the horizontal and vertical angles depending on calculated movements of the point and of the stations. This is to try to reduce potential errors linked to the sighted point moving on the ground (there is no search of a prism centre as with the usual use of a RTS, so a movement of the ground or of the RRTS would lead to a different point being sighted for unchanged horizontal and vertical movements).

In Amsterdam (Netherlands) over 82 total stations (See Figure 2) are used to measure surface movements above the tunnel boring machine during the construction of the metro line, both with conventional RTS and with RRTS. Due to the quantity of points measured: 5320 RSP for RRTS and 5820 prisms for RTS, and the delivery period of one hour, the standard method is used to comply with the client's requirements.

In addition to RRTS and RTS a network of manual levelling benchmarks on buildings, quays and on the ground was set up. The 3590 levelling benchmarks confirmed the consistency between the precise levelling and the RSP movements. The precision obtained was better than ±1 mm on the RSPs.

The Mesh Method

The mesh method uses a number of RSPs around the point of interest to smooth and eliminate automatically any surface irregularities, through a geographical statistical treatment of the measurements. This method is therefore more complex, but is has been well proven in practice since 2005.

In Toulon (France) during the construction of the south road tunnel a network of 1830 RSPs have been measured over roads and pavements along the tunnel excavation from 36 total stations fixed positions (see Figure 3). They allowed the measurement of cross sections every 9 meters, larger or smaller depending on the urban environment and to deliver data every 2 hours.

An external control using traditional precise levelling on benchmarks was performed to validate the results with a precision about ±0.5mm.

Figure 3. Reflectorless Robotic Total Station in Toulon.

Table 1. Comparison between the standard and the mesh method

Method	Pros	Cons
Standard Method	• Fast: Time depends on model of total station used: approximately 5 to 10 seconds per reflectorless measurement point. • Simple.	• Slightly lower precision, approximately ±1mm. • Risk of false reading and even false trends depending on the state of the surface.
Mesh Method	• Very high precision in the order of ±0.5mm. • Numerous security quality checks.	• Rather slow process, each point of interest requiring between 30 seconds and 1 minute of sightings.

Pros and Cons

The advantages and drawbacks of each method can be summarised as shown in Table 1.

Conclusion

Real time monitoring with RTS has demonstrated the value of this method for many years. Thanks to the improvement of the range and the repeatability of the laser beam, monitoring of surface deformation with RTS using reflectorless measurements (RRTS) has become reliable, precise and very helpful as an early warning system, detecting movements and trends 24 hours a day.

Generally RRTS is slightly less precise and the range is shorter than the RTS method but for safety purposes it is an ideal solution for dangerous sites and an alternative to levelling measurements with a high frequency of readings.

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The GROUT LINE

Grout Line

Paolo Gazzarrini

Overture

Here we are at the 25th issue of the Grout Line and, as promised, Dr. Lombardi's answer to Dr. Bruce's article, published in the June 2011 issue, has arrived quickly. Following are the comments of Dr. Lombardi (Studio Lombardi- Minusio-Switzerland-info@lombardi.ch)

Some Considerations on the GIN Grouting Method

Introduction

In the recent paper titled "Rock grouting for dams and the need to fight regressive thinking" [1] Dr. Eng. Donald A. Bruce presented some general comments on the grouting methods in use, subdividing them into classes of "old", "new" and "regressive" methods or concepts.

Many observations that he made are quite interesting and can be entirely shared. Others are difficult, if not impossible, to understand and seem to be based on a lack of information or on some misunderstanding. It is my feeling that a few comments on these points are due and could be of some interest. The comments will be restricted to the pure grouting activity, leaving aside the problems related to the drilling methods.

On "Old" Methods

It can be entirely agreed that the methods used in the USA from 1920 to 1980 - and in some cases also to the present time - that is, since about the time of the grouting of the Hoover dam to today - should be finally changed.

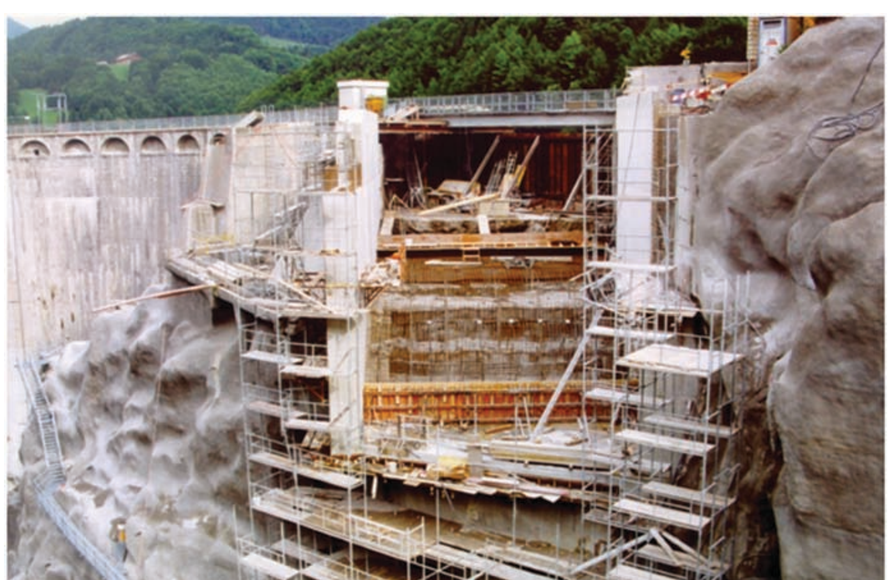
The main points of these old methods are:

- the drilling of vertical holes to a target depth;
- the "one row" curtain;
- the relatively low grouting pressure;
- the use of "thin" mixes;
- the "thin to thick" mix grouting method;
- the use of drillings of higher order holes to sometimes "ridiculously close centers"
- the use of thin mixes injected in karstic cavities.

It is completely agreed that these "old ways" contain major flaws and have to be changed!

The "New" Method

The "new method used in the USA" is claimed to have improved the grouting procedure in a number of important points. Mainly:



Montsalvens Dam (Canton Fribourg, Switzerland), built in 1920. One of the first arch dams in Europe.

- consideration of the importance of the “pressure filtration coefficient”. This coefficient corresponds essentially to the old French “presso-filtration”;
- use of various chemical admixtures and not of water to the grout to reduce the cohesion (and the viscosity) of the mix, ensuring an enhanced penetrability;
- introduction of computer-based systems of monitoring the grouting;
- use of new methods of scrutinizing the wall of the drill hole;
- “curtains must have at minimum 2 rows of holes”;
- use of inclined holes to upstream;
- declaring and measuring of the residual permeability;
- definition of a “stage refusal”.

In general one can share the principles mentioned under this title, with, however, a few comments which will be presented later on.

The “Retrogressive Principles”

For sure, there are still existing cases where old methods are used again, in spite of the problems and flaws they present. So e.g. the use of different mixes from thin to thick.

On the contrary it is quite difficult, or even impossible, to understand why the GIN method is declared as “retrogressive”. The following considerations need to be made:

- The commentary that the GIN method was developed to “assure for the client a certain standard of care and quality … on projects … in remote areas … or by contractors with limited experience and expertise” is difficult to understand, unless one considers, for example, Austria and Switzerland as remote areas and their contractors as having a limited experience and expertise!
- Since the GIN method theory was first elaborated, it has been declared that a grouting work should be “designed” (engineered) not “specified”. This appears now and finally to be quite a “new” concept.
- From the beginning the GIN method has been based on the use of a unique grout mix, the “best one”, - obviously among the ones that are available in practical terms - (cost, availability of certain material and so on). This is now claimed to be a “new” concept.
- The definition of the best mix must be based on two aspects: the “best for grouting” and the “best for the final result to be achieved”. Therefore some compromises may be required in special cases. Obviously throughout the decades the “best mix” changes due to technological progress (e.g. due to available chemical admixtures), but this does not change the principle of the GIN method.
- The continuous monitoring and representation of the data from the grouting of any stage such as pressure, flow rate, volume and penetrability of the grout was always one of the principles of the GIN-method. Obviously, the way to do it did follow the continuous progresses of the electronic equipment available. In some of the new ones the GIN value itself is directly shown on the screen and all the required graphs automatically produced. Now this should also be considered a “new” concept.
- Generally speaking, it is also clear that some improvements were implemented from time to time in the GIN method, in order to keep it continuously “new”.
- It is also felt that the definition of a maximum pressure is unavoidable at least to define the pumping equipment to be used. Thin is not a “new” nor an “old” concept.
- The definition of a maximum take is also to be considered necessary to avoid excessive losses.
- In any case the three limits (maximum pressure, intensity and volume “limit”) must be the result of previous grouting tests not values arbitrarily “specified” a priori. They should be changed if the rock conditions are locally different from the general assumptions made.
- It was shown, by theoretical considerations as well as by events during grouting, that in given conditions the hydro jacking is a function of the “grouting intensity”, that is of the energy pumped in at any stage. The definition of a number GIN is the logical consequence of this fact.
- By the way, it was always clearly stated that the numerical values indicated in the papers on GIN are not “recommendations” but are just “naming” for typical cases which represent the average of a number in particularly successful grouting works. These statistics allow us to recognize the type of grouting carried out (low, medium, high, etc.). In fact, it would be quite difficult to recommend, at the same time, an extremely low as well as an extremely high grouting intensity.
- It is well known that Prof. Ewert doesn’t like the GIN method. Indeed, when in certain cases the grouting causes too many occurrences of hydro-fracturing, this means simply that wrong values were chosen by the user, often because the required preliminary tests were not carried out.
- If he found that in certain cases the volume limit is too low to fill the voids of certain rocks, this means again that the limit selected was too low, maybe for the same or any other reason. In any case, the GIN method is not intended to fill karstic cavities!
- Indeed, these and other misunderstandings of the GIN method are due to the fact that many people are mostly looking for fixed norms and specifications and are not very interested in understanding ideas and concepts.
- Furthermore, the basic idea of the GIN, in limiting the intensity (p•V), is to allow high pressure in order to increase the reach of the grout where the penetrability and thus the take are low and, at the same time, to eliminate - or at least reduce - the cases of hydro-jacking when the take at low pressure is too high. It must be repeated that the risk of hydro-jacking is the highest by the combination of high pressure with high take. Should the hydro-fracturing (indeed hydro-jacking) still be too frequent, then the GIN value

should be reduced and adapted to the actual conditions of the rock mass.

- By the way, it is strange to have to take notice that the GIN is criticized by Dr. D. Bruce because of volume limits considered to be too low and at same time because the method allows one to overpass the same limits due to the fact that they are considered to be a point of decision not an absolute limit. The GIN method has also been blamed for having adapted, at some date, the interpretation of the limit for the maximum take. Maybe, this was just an improvement of the method in order to remain "new"! In any case only the "last version" of the method needs to be discussed.
- All this is independent of the fact that the three limits are to be defined by the designer on the basis of preliminary grouting tests and not "specified" a priori.
- It has to be noted that in many cases a wrong use of the method was carried out, leading to poor results. This fact apparently authorized a number of authors to accuse the GIN method of not working properly and obviously at same time to excuse the engineer who did not understand it. A typical, many times repeated case, is the one of karstic rock. It should be finally clear to everyone that the method is designed to be used in "solid, fissured rock masses", not in karstic nor in too weak rock or loose ground.

Old Concepts Still in Use in the "New" Methods

A number of very old concepts are still in use in the so-called "new" methods and, in the opinion of the writer, should finally be changed:

- The first concept is the expression "refusal". In the way it is generally used, it suggests that the rock mass would "refuse" any additional grouting. Indeed it is the designer who refuses to use higher pressures, or possibly the pump that does so! Never the fissured rock. Therefore, a different word should be used to describe the fact that the specified, designed, or arbitrarily selected pressure value was reached while the flow rate is nil.
- A second old habit should also be updated. To stop the grouting procedure it is usual to maintain the prescribed pressure for a certain duration until the flow rate falls below an "arbitrarily" defined value. By experience it is much more efficient to overpass slightly the given pressure by - let's say, 5% or 10% - to stop the pump and to observe the falling of the pressure at flow rate nil. According to the pressure arrived at after a short duration, the operation can be stopped or continued. Obviously, a certain tolerance of a few percentage points on the final pressure should be allowed. In many cases the procedure appears to be quite simple and effectively time saving. At least the two ways of defining the ending of the grouting should be accepted.
- It is felt also that in a number of cases some attention should be paid to the grain size of the cement.

Also the concept, about one century old, of having to limit the grouting pressure to the weight of the overburden should finally be abandoned because, while a limited heave can generally not be avoided, depending on the conditions, pressures many times higher (up to 3 times) can be used at no risk of substantially heaving the ground.

All of this is just to recall that the newly used concepts are not always as "new" as claimed or as they should be.

General Comments on Grout Curtains

It has to be taken into account that with the depth below the ground the conditions are changing due to the increased stresses in the rock mass; so the permeability generally decreases. Also due to hydraulic considerations, the requirements to the grout curtain are decreasing with the depth because of the longer paths for the water to flow from upstream to downstream. Therefore one should not prescribe as a general rule a number of rows, but instead, adapt its number to the depth.

Additionally, the constant length of the grouting stages generally chosen for all boreholes of the curtain (e.g. 5 m), is the result of an "old concept" which can be improved by increasing the length of the stages with the depth. (No indication or examples are given here in this respect in order to avoid that they would be understood as firm numerical "recommendation"!)
This is a "new" concept not yet considered in the "new" methods, except in the GIN one.

The opinion that the depth of the curtain should not be fixed a priori but adapted to the geotechnical conditions is entirely shared by the writer.

The actual permeability across a grout curtain is always quite difficult to be measured and defined with precision. The relatively high pressure used by the Lugeon tests (not to be compared directly with the grouting pressures but with the reduced grout pressures at some distance from the borehole) may cause some damages to the curtain. The question thus arises whether the decreasing of the takes from borehole to borehole and to the control holes should not be used as a criteria to better define the results of the grouting curtain instead of using water pressure tests with possible damages.

Additionally, it has to be questioned whether in many cases a distinction and a difference must still be made between "consolidation" and "grout curtain grouting" or whether the definition at a unique, comprehensive treatment zone should not be preferred.

Conclusions

From the above comments it results that a number of improvements in grouting rock masses are still possible in matters of "new concepts" not yet considered in the "new" methods.

In any case it appears quite clearly that, for the time being, the GIN method is by far not outdated and that it can in no way be presented as retrogressive. This could obviously happen in the future, should radical improvements of any kind take place in the field of grouting.

For the time being, the GIN method can apparently claim for itself to be a "new" or even the "newest" one.

In front of these facts, it could be interesting to hear on the basis of which considerations the GIN method has been defined as "regressive" or "retrogressive". This is independent of the fact that the method is "disturbing" a number of people and this for unknown reasons.

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...... to be continued?

With that I close this issue, hoping that you have had a nice summer and I remind you that, if you have additional comments or interesting grouting stories or case histories, you can write to me: *Paolo Gazzarrini, fax 604-913 0106 or paolo@paologaz.com , paologaz@shaw.ca or paolo@groutline.com. Or tweet me @groutline*

Ciao!

WASTE GEO Technics

Characteristics of Municipal Solid Wastes and Landfill Disposal in China

Tony L.T. Zhan, Y.M. Chen, G.W. Wilson and D.G. Fredlund

Introduction

The management of municipal solid wastes (MSW) is one of the major problems associated with rapid urbanization in China. Since 1980, the rapid growth of China's urban population has resulted in the generation of an exceptionally large amount of MSW (Fig. 1). According to the World Bank (2005), China produced 195 million metric tons of MSW in 2005 and became the world's largest generator of MSW. At present, the annual generation of MSW is 245 million tons with an annual increase of about 7%. The amount of generated MSWs in China in 2030 is estimated to be as much as 480 million tons.

There are four ways in which MSW is generally disposed (i.e., landfill, incineration, composting, and recovery). As of 2007, the first nationwide investigation of pollution sources in China (MOEP, 2010) indicated that 90.5% of collected MSW is disposed of in landfills, 8.1% is combusted, and less than 2% is composted. There were over 800 landfills in 661 cities in China as of 2007. These landfills received 150 million tons of MSW. Placing MSW in landfills will likely remain the dominant disposal method in China as it is cost-effective and can accept mixed waste without requirements for separation.

In the past two decades, landfill technology and practices in China have improved significantly due to the enactment of a series of new regulations, policies, and technical standards (Dong, 2009). Landfill technology in most cities has shifted from simple dumping to controlled landfilling. However, there is still a gap between prescribed standards and engineering practices mainly due to a lack of financial resources, research, education, early planning, site-specific design, and sound landfill operation practices. The rapid increase in the volume of MSW has outstripped service demand. Further developments in landfill technology are being challenged by environmental and geoenvironmental concerns. There have been issues related to landfill failures, soil and water pollution, emission of greenhouse gas, shortage of land for waste disposal, and a lack of financial assurance related to closure and post-closure care. In addition, there have been technical difficulties associated with the recovery of landfill gas and the reuse of closed landfills.

This article presents the composition and characteristics of MSW, the current state of MSW landfills, and the challenges related to further development of landfills in China.

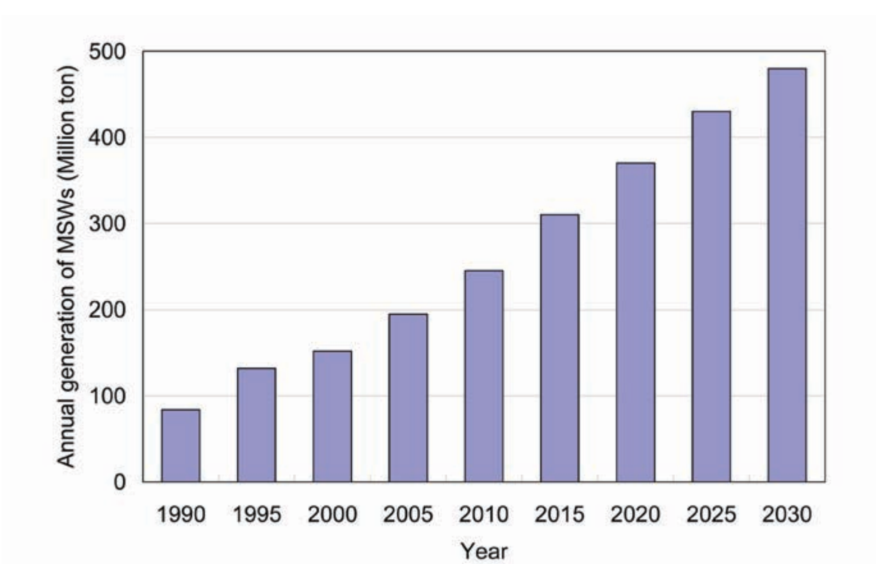

Figure 1. Trend of MSW generation in China from 1990 to 2010 and estimates of MSW generation up to 2030.

MSW Composition and Characteristics

An understanding of the composition and characteristics of MSW is important for the planning, design, and operation of landfills. MSW typically consists of food and vegetable wastes, paper products, plastics, textiles, wood,

Table 1 Comparison of waste composition among China, India, Korea, Singapore, UK, and USA generated in 2000 (Unit: %)

Country	Food, vegetable	Dust, cinder	Paper	Plastic, textile, wood, rubber	Metal, glass	Others	Water content
China	43.6	23.1	6.7	16.7	3.4	6.5	52
India	41.8	40.3	5.7	8.2	4	0	42
Korea	24.6	NA	25.8	NA	13.5	NA	39
Singapore	23.5	17.1	21.6	11.1	24	2.7	35
UK	20	12	34	21	15	0	32
USA	15.3	10.9	29.8	29.4	12.7	1.9	30

cinder, and soils. The proportion of each component varies from one region to another. Even within a single region, changes in living standard, legislation, seasonal factors, and pre-treatment and recycling activities may result in changes in the waste stream over time. Figure 2 shows the changes in the composition of the MSW collected in Suzhou, China, between 1990 and 2006. The content of each component was measured and calculated on a wet-weight basis. It can be seen that there has been a significant decrease in the cinder content between 1990 and 2000. The change was the result of an increased use of natural gas for cooking. At the same time, there was an increase in recyclable content and food and vegetable wastes between 1996 and 2000. Since 2000, the change in the composition of MSW has been relatively small.

Table 1 shows a comparison of waste composition among China, India, Korea, Singapore, UK, and USA as of 2000. The MSW in China and India contain much more putrescible organic wastes (i.e., kitchen food and vegetable wastes which account for 40-50%) than the MSW generated in Korea, Singapore, UK, and USA. The content of mineral materials (i.e., cinder, dust, concrete, etc.) in China and India is also higher than that in UK and USA. These differences are likely attributable to the differences in cooking styles and the living standard among the countries. The MSW in UK and US contains much more recyclable matters (particularly paper products) than the MSW found in China and India.

The characteristics of waste composition in China result in particular properties of the wastes. First, the initial water content of the MSW collected in China ranged from 40 to 60% (by wet mass), which is much higher than that of Europe and North America. The high water content is mainly due to the high content of food and vegetable wastes in China.

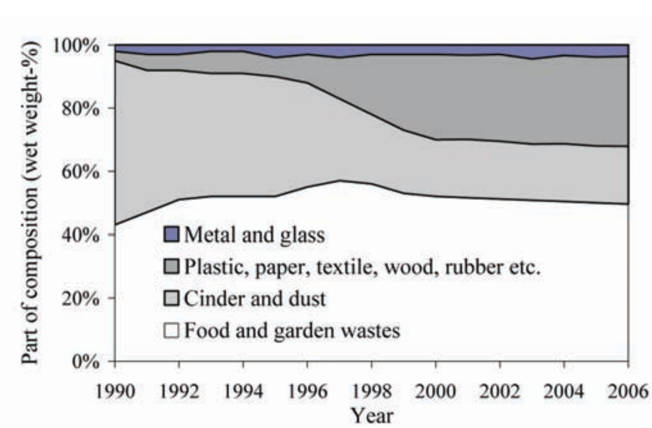

Figure 2. Change in the composition of MSW generated in Suzhou, China.

Second, the high organic content in the Chinese MSW means that there is more biodegradable material for decomposition in the landfill. Waste decomposition tends to result in greater loss of solid mass, higher production of gas and leachate, and greater time-dependent compression. The hydraulic and mechanical properties of the wastes in China are significantly different from the properties of the low organic content MSW in Europe and North America. Laboratory measurements of MSW generated in China indicate that 300-400 L of gas (i.e., mainly methane and carbon dioxide) could potentially be produced per kilogram of dry waste.The decomposition-induced compression of Chinese MSW was measured as 25% of the waste thickness when it was subjected to a vertical load of 50 kPa. Shear strength measurements on China's MSW showed a mobilized cohesion of 23 kPa and an internal angle of friction of 10° for the fresh wastes. Wastes with a fill age of 11 years showed a cohesion of nearly zero and an angle of internal friction of 28° (Chen and Zhan, 2007).

Third, the high water content in the MSW in China tends to result in higher leachate production at a landfill, as compared to the MSW from Europe and North America. Field observations at many landfills in southern China indicate that the quantity of daily leachate production is more than 30% of the daily dumping mass of wastes. In addition, the high content of food and vegetable wastes in China tends to result in a low pH value and high mass loading in the leachate generated at the landfills (He, 2009). Field and laboratory measurements indicate that the leachate produced at landfills in China generally possess a much higher mass loading (Total Organic Carbon (TOC): 9,000-13,000 mg/L; Chemical Oxygen Demand (COD): 40,000-80,000 mg/L) and a lower pH value (4.8~5.4), compared to North American landfills (TOC: <8,000 mg/L; COD: 15,000-35,000 mg/L).

Landfill Disposal of MSW

Simple Dump

Landfilling is the dominant disposal method of MSW in China. The history of landfilling in China is about 10 years behind countries in Europe and North America. Prior to 1990, simple dumping of MSW prevailed in most cities of China with generally small volumes of MSW that was distributed among several locations in each city. For example, there were over 1,000 MSW dumping locations distributed in the city of Beijing. Most of the simple dumps have been closed, but controlled closure measures have not always been followed.

First-generation Controlled Landfills

After the first *Technical Standard for Sanitary Landfill of MSW* was issued and put into action in 1988, landfill technology and practice in China shifted from simple dumping to controlled landfilling. In the 1990s, so-called first-generation controlled landfills were built in many cities around China. During this period, there were over 800 landfills, about 45% of which were designed as controlled landfills. The first-generation of controlled landfills made use of vertical barriers and toe drains for leachate control. Most landfills were located in a valley or canyon. The low permeability bedrock generally formed the shape of a "dustpan" with an opening downstream from the landfill. Vertical barriers, extending to the underlying fresh bedrock, were installed at the downstream opening. The vertical barriers and the bedrock were expected to form a closed barrier system against the leachate transport to the surrounding environment. The vertical barriers commonly used in China consisted of a plastic concrete cutoff wall in the soil deposit and/or grout curtain in the weathered rock. There is still uncertainty and debate regarding the long-term performance of this type of vertical barrier. There are variations in the geology and barrier design that need to be taken into consideration. Toe drains have been used for leachate drainage at the first-generation controlled landfills. The toe drains were usually installed at the retaining dam downstream of the landfill. Field observations have shown that there can be serious clogging problems for the toe drains after the landfills have been in operation for a while. The clogging of toe drains combined with poor surface water management has resulted in the continual accumulation of leachate, and hence, a high leachate mound within the landfill. Figure 3 shows the distribution of a leachate mound within the Suzhou landfill. The plot of the leachate mound was deduced from field measurements of pore pressures and unsaturated-saturated seepage modeling (Chen and Zhan, 2007). The maximum height of the leachate mound in the landfill bottom is 15 m. A substantial perched leachate mound was also observed on the intermediate soil cover.

Control of landfill gas emission was not widely implemented at first-generation controlled landfills. Few of the landfills were equipped to generate energy or recover landfill gas. Additionally, many landfills had poor management practices: landfill operations generally involved poor planning of waste placement, poor compaction of waste piles, use of soils for daily and temporary covers, and poor management of surface water. As a result, the daily leachate production at the first-generation controlled landfills was commonly more than 30% of the daily dumping mass of wastes, which was much greater than expected. The high leachate production tended to overwhelm the leachate storage pond and treatment system. Stated another way, the first-generation controlled landfills operated at a low level. There is a lack of detailed data, but it is quite clear that some landfills have had an adverse environmental impact.

Second-generation Controlled Landfills

In the 2000s, many of the first-generation landfills had reached their service design capacity. New and expanded landfills are being built in many cities in accordance with the revised regulation and standards, including the *Technical Standard for Sanitary Landfill of MSW* (CJJ17-2001, CJJ17-2004) and *Pollution Control Standard for MSW Landfills* (GB 16889-1997, GB 16889-1997-2008). The new landfills are called second-generation controlled landfills. The functionality of this generation of landfills is quite similar to that of modern landfills in North America. A composite liner system, consisting of a basal sealing liner and a leachate drainage and collection system (LDCS), is commonly used for leachate control. China's landfill technology and capacity has increased significantly to meet the service demand associated with rapidly increasing waste generation. The daily filling capacity for landfills in large cities is commonly over 3,000 ton/day with a designed service period of at least 20 years. For example, the Laogang landfill site in Shanghai has a daily capacity of 5,000 tons with an estimated service period of 45 years. With an increase in capacity, the number of controlled landfills in China decreased significantly from 571 to 366 from 2001 to 2007.

Many second-generation controlled landfills have taken measures to reduce landfill gas emission. Landfill gas drainage and/or its collection have been implemented at most controlled landfills. Over 20 landfills are equipped with gas recovery facilities, and the total generation of electric power is about 50 MW. Several landfills have succeeded in the application of projects adhering to the Clean Development Mechanism (CDM) under the United Nation Framework Convention on Climate Change. Landfill management practices have also significantly improved. Many landfill operators devote considerable effort to develop waste phasing schemes and surface water control measures to separate precipitation from leachate. Al-

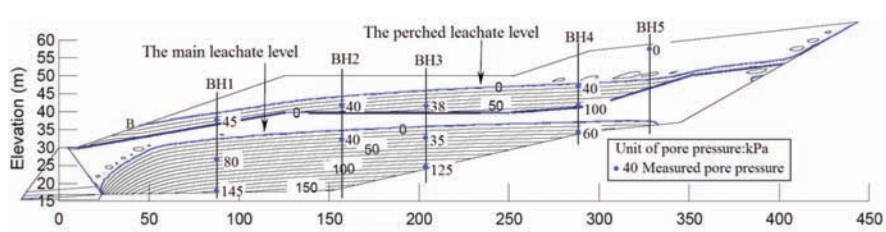

Figure 3. Pore pressures measured in the field and distribution of a leachate mound predicted by numerical simulation.

ternative materials such as low-density polyethylene (LDPE) geomembranes are widely used for daily and interim covers. The improved operation practices at second-generation controlled landfills have certainly resulted in a reduction in leachate production.

Leachate production in the southern cities of China is still more than expected due to the high initial water content of the MSW. High leachate mounds are still observed at many of the second-generation controlled landfills in China. Field evidence indicates that the development of high leachate mounds is mainly attributed to the clogging of the bottom LDCS. Maintenance of LDCS such as retro-flushing of collection pipes during landfill operations has not been practiced in China. It appears that the LDCS at landfills in China are more susceptible to high leachate production as well as heavy mass loadings of leachate. The high leachate mounds in the landfills tend to cause slope failures in landfills, increase risks of environmental pollution, and inhibit landfill gas collection. High leachate mounds within landfills appear to be one of the major technical difficulties discouraging landfill gas recovery in China.

Technical Challenges Associated with MSW Landfills

Landfilling will remain the dominant disposal method for MSW in China for the foreseeable future. Further development of controlled landfills in China will encounter the following challenges:

1. Stability and safety of the landfills with a continuous increase in height: Many landfills in China have reached a height of 60 m and will reach a height of over 100 m in the future. In recent years, there has been an increase in the number of landfill failures, including in Chengdu, Shanghai, Shenzhen, and Xian. Most of the landfill failures were triggered by a high leachate mound. With an increase in landfill height, the stability problem will become more critical if there are no effective solutions to control high leachate mounds.
2. High leachate production and treatment: High leachate production and high mass loadings result in expensive leachate treatments. Many landfills are not able to afford the high cost of treatments required to meet the new leachate discharge standard in GB 16889-2008.
3. Land shortage and increased landfill capacity: Many cities lack land for disposing increased masses of waste. Increasing the capacity of existing landfills is an effective but temporary solution. This is usually done by expanding the existing landfill, increasing the slope angle, and accelerating waste decomposition and settlement. Landfill stability should be assured when implementing these measures.
4. Recovery of landfill gas: Collection and recovery of landfill gas can reduce greenhouse gas emissions and provide energy (and income) for landfill operation. The efficiency of gas collection in China is lower than 40% due to the high leachate mounds in landfills. Breakthrough technologies addressing this difficulty will encourage the recovery of landfill gas at the 366 controlled landfills in China.
5. Control of soil and water pollution by leachate: Different degrees of underground pollution have been reported at many landfills in China. The major challenge of this issue lies in the durability of barriers, especially when the barriers are subjected to a high leachate head. In addition, quality assurance and control for the construction of barriers should be made more stringent.
6. Closure, post-closure care, and reuse of closed landfills: There are thousands of simple dumps requiring proper closure as well as long-term post-closure care. Financial assurance for this activity has not yet been considered in municipal budgets. Financial sources for closure and post-closure activities may come from land reuse of closed landfill sites. Land reclamation activities will encounter difficulties associated with post-closure settlement and environmental impacts from the landfills.

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the Water in the Soil – Part 4

Bill Hodge

What I want to do in this article is tidy up a few things about liquefaction before I move on to trying to sort out pore pressure generation within a saturated aggregation of soil particles. First, I'll suggest why silts do not seem as prone to liquefaction as sands. Then, I will look at some good triaxial testing to see if there is any support, or conflict, between this hypothesis and those laboratory findings. After that I'll touch on the possible different effects earthquake induced shear waves and surface waves might have on liquefaction behaviour and structural responses.

Can Silts Liquefy?

It is in attempting to answer questions such as this: "Can silts liquefy ?", that the utility of a new physical model of two-phase soil behavior can be evaluated. So here I'll attempt to use the L-factor and the "soil" components of the drag force to see if I can explain what's special about silts when it comes to situations where sands would be expected to liquefy and silts seem not to. With this in mind I'll use the pieces put in place already in earlier articles to see how this question might be answered.

In Figure 6 (Part 2) I suggested that particles of a size which can reach their v_T [Terminal velocity] within a fall distance of less than 29% of their diameter are inherently vulnerable to liquefaction. The reasoning behind this is that this is the distance uniform spheres move downward while changing from the loosest packing arrangement to the densest. Silt sizes are all well below this red line, so the implication is that silts are extremely likely to liquefy when going from a loose to a dense packing.

As can be seen in Figure 8 (Part 3), for velocities at v_T , the L-factor is zero over the full silt size range. That means, according to this model, that the Pressure component of the Drag force plays no part in silts under liquefaction conditions, and hydrodynamic resistance to relative velocity [particle movement] is fully accounted for by the Bearing component alone. It implies that when a silt size particle falls enough to have liquefied it does not result in the generation of pore water pressure, as a sand in similar circumstances would, but instead results in a viscous response which I have equated to bearing resistance of a cohesive soil. So, rather than liquefying, a loose silt deposit would tend to consolidate. And since there would be no generation of excess pore water pressure to produce a critical gradient, there could be no manifestation of concentrated venting through local weakness at ground level.

Along this line of reasoning one might wonder if the same way of looking at the behaviour of fine particles might have some involvement in permeability, consolidation and creep.

Before leaving silt there is a point I want to make: The abrupt discontinuity I have shown for the L-factor at Re = 0.6 doesn't seem quite right to me. There isn't the gradual change from one mode of behaviour to the other that I expect to see in physics where there aren't very different material properties across the boundary. What I'm inclined to think is that this discontinuity is perhaps because our colleagues in Fluid Mechanics didn't, or couldn't, define the values accurately in this area of conjunction. But, as I don't have the ability to come up with my own values, I will settle for theirs and trust that someone else may follow up on this. I suppose I'm basically expecting to see the gradual sort of changes we get in silt and sand sizes as we cross the 200 mesh sieve – nothing startling.

A final word about silt liquefaction: It has been reported that the loess flow slides which resulted form the great 1920 Kansu earthquake in China involved silt liquefied in air. But with no water present, "liquefied" is hardly the word for it – perhaps "fluidized". In any event I think this behaviour is consistent with the above understanding since the viscosity of air is less than one fiftieth that of water. In consequence, the air would not have the bearing capacity to prevent the coarser silts from falling downwards, thereby leading to generation of increased pressure in the pore air beneath them. And I imagine that if such pneumatically charged air were entrapped within the body of the sliding mass it would add mobility to the motion.

Triaxial Testing

James K. Mitchell

Sometime after Professor Mitchell moved from Berkeley to Virginia Tech he was kind enough to write me with a critique of some of these ideas. While he was encouraging about my bearing capacity analogy for small particles he was troubled about what seemed to be a logical inconsistency between laboratory results and saying that motion was the source of pore pressure. The point he raised was: In an undrained triaxial test, when there is an increase in pore water pressure recorded during a test, how could that pressure increment still remain after specimen straining was stopped, if motion was the only reason for pressure generation in the first place? I believe this is a question which is likely to arise again, so I feel the need to address it now.

One response might be just to point out that what's going on inside the membrane of an undrained test is directly analogous to stopping the piston's advance in a hydraulic cylinder which is not leaking. But I think it is more useful to look at the triaxial apparatus itself. The cell pressure is transmitted across the membrane to the soil particles and also to the water inside the sealed specimen enclosure. The force radially inwards at any stage is equal to the membrane surface area times the cell pressure. The outward balancing reaction to this force is the summation of the pressure increments on each particle in contact with the membrane, plus the pressure on the remaining area of membrane in contact with the pore water. During the test, while straining is being imposed, these forces and pressures change depending on how the soil-structure dilates or contracts. But the moment straining is halted these values are "frozen", and apart from any subsequent creep of the soil-structure which might occur within the membrane, I can see no reason for the pore pressure at the end of the test to diminish in value. It just sits there.

Gonzalo Castro

Dr. Castro's research work at Harvard must surely rate amongst the best and most significant geotechnical laboratory work yet performed. Figure 10 is a photocopy of his "Fig 22" from his work published in 1969 as *Harvard Soil Mechanics Series No. 81*. It shows the stress-strain record of a consolidated undrained triaxial test performed on a specimen of his sand type B. Liquefaction was brought about by monotonic axial compression. Here u_d is the pore pressure change induced by application of deviator stress σ_d. The axial load was increased gradually over a 14 minute period by adding dead load increments. When the load exceeded the strength of the soil-structure it failed in an instant. One thing that I find particularly informative here is that approaching the point of failure the pore pressure is only about half its final value; it is only after failure of the soil-structure that it rose to about 93% of confining pressure. In fact I believe the pore pressure increase prior to liquefaction may be attributable to changes in the proportions of the membrane interface with particles and with water as the soil-structure tries to accommodate the increasing load. It is also apparent on this data record that much of the pore pressure increase happens while the particles are collapsing.

Yoginder P. Vaid

The work done at the University of British Columbia under Professor Vaid's guidance in the early 90s was most useful and enlightening to me. Consolidated undrained triaxial testing of Fraser River sand showed quite clearly that whereas this uniformly graded natural material was dilative in compression at even the loosest (pluviated) densities, it could be brought to liquefaction at relative densities up to 40% when subjected to axial extension. The importance of this radically different behavioral response to stress path will be discussed below with respect to wave forms created during earthquakes.

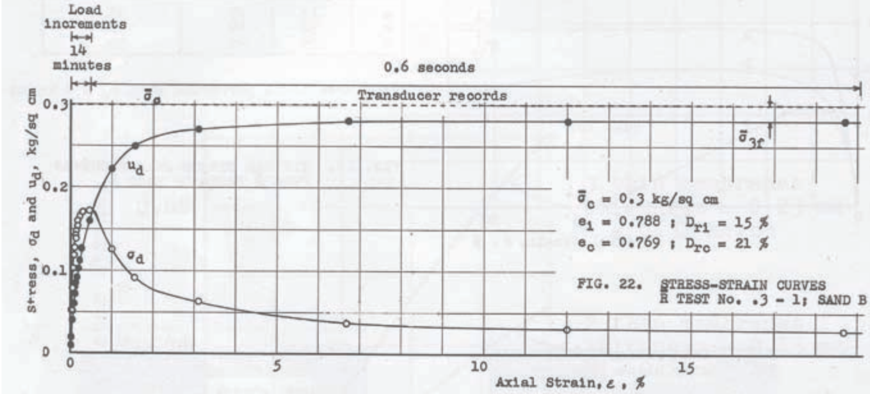

Figure 10. Castro's Fig. 22 from his Harvard publication.

Triaxial Results in Terms of Fall-to-Diameter Ratio

In Part 2 the Fall-to-Diameter ratio [F/D] was introduced as a numerical criterion for assessing the opportunity of individual spheres to reach v_T based on their diameter, in comparison with the amount of space available for them to fall through water as their packing arrangement changed from a loose state to a dense state. In fact what I used were the maximum and minimum void ratios (e_{max} and e_{min}) of idealized arrays of uniform spheres. There I gave the value of F/D as 0.29, which is the numerical value of the exact mathematical solution, $1\text{-}1/\sqrt{2}$, for this change in position. Another way of arriving at this same value is to consider the ratio of downward displacement of the centre of gravity of a saturated mass of uniform spheres per unit height of the initial assemblage. This can be expressed as $(e_{max} - e_{min}) / (1 + e_{max})$ which is also equal to $1\text{-}1/\sqrt{2}$, since $e_{max} = 6/\pi - 1$ and $e_{min} = 6/\pi\sqrt{2} - 1$. Taking advantage of this correspondence I decided to plot the results of both Castro and Vaid in terms of what their specimen void ratios suggested about this type of equivalence to F/D, by replacing e_{max} in this relationship with the loose void ratio at which the specimen was prepared.

Figure 11 is based on the same computational approach used to make Figure 6 in Part 2 of this series. Here it has been drawn to a larger scale since it is only sand sizes I want to look at. The heavy black line labeled 99% identifies the ratio of the distance a spherical particle must fall in relation to its diameter

Table of Equivalent "Fall-to-Diameter" Ratio Limits (values plotted as rectangles on Figure 11)					
Sand Type		Size, mm		Fall ÷ Diameter	
Symbol	Source	D85	D15	Upper	Lower
A	Salt Lake earthfill	0.304	0.130	0.254	0.220
B	Ottawa Banding	0.217	0.108	0.183	0.127
C	Huachipato Beach	0.452	0.159	0.197	0.163
Y	Fraser River	0.325	0.215	0.134	0.103

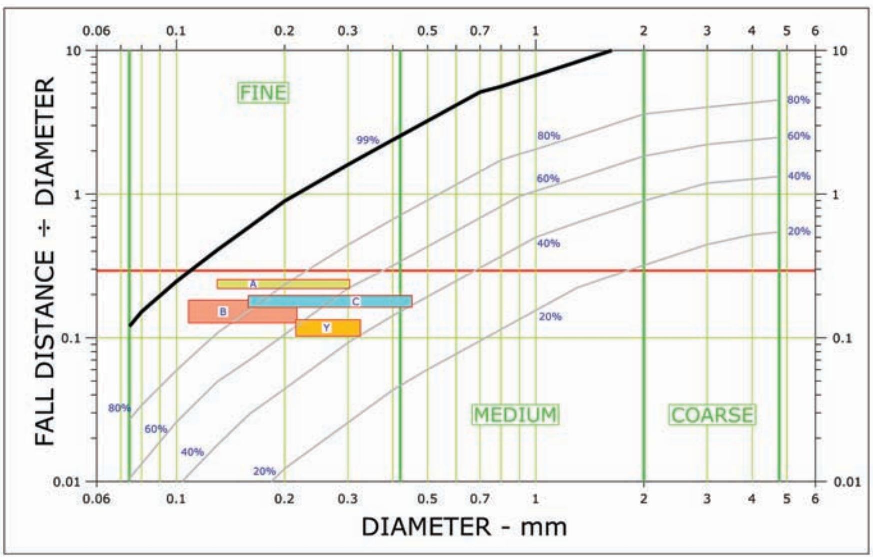

Figure 11. Castro and Vaid results on Fall/Diameter plot.

[F/D] in order to transfer 99% of its buoyant weight to the water it is falling through. Similarly the other percentage labels are for lesser amounts of weight transfer to the water.

The three rectangles, labeled A, B and C are thus derived from Castro's three sets of triaxial tests over the range of densities where the specimen liquefied under monotonic loading. The letters are the same as those used by Castro in naming the three different sands he used. The rectangle labeled "Y" is for the extension tests which resulted in liquefaction (steady state stress-strain) of the Fraser River sand Vaid used.

The vertical sides of the rectangles are at the D_{85} and the D_{15} gradation sizes for each particular sand type. The upper and lower horizontal sides of the rectangles cover the range of void ratios at which specimens were made, and which resulted in liquefaction failures. These values are listed in the table, where it can be seen that soils fall into the category of fine sands and the equivalent F/D range lies between 0.1 and 0.25. Here it is necessary to point out that although some of these numbers are quite close to the red-line value of 0.29, and might be considered as providing some support for this proposal, this is not the case. Instead, they need to be compared with the values along the black/grey curved lines representing the percentage of weight transfer to the water.

As may be seen from this mode of presentation the losses in effective particle weight range from 40% to 90% with the average being somewhat less than we would expect during a liquefaction failure. I believe a better interpretation of this plot requires consideration of the Crowding-factor [K], since the % transfer lines are based on single particle responses, whereas here we are for the first time dealing with the soil mass. "K", which will be subsequently introduced and developed in Part 5 of this series, is essentially an amplification factor on relative motion. As such it has the effect of reducing the amount of fall necessary to achieve a particular level of weight transfer, and therefore, should bring the curves more into line with these laboratory results.

Shear Waves and Cyclic Loading

Computer programs which deal with the transmission of shear waves through soil, such as SHAKE, have proven very useful (and surprisingly accurate) in predicting how tall buildings move/sway about in response to earthquake vibrations. These programs are based on how small strains of different frequencies would be either amplified or attenuated as they pass through a stable/intact soil-structure. I doubt if the original authors would have condoned their use for soils which were strained to the extent that they were collapsing. However that may be, what is known for sure is that shear waves cannot pass through a fluid, and this presents a problem when dealing with soil we expect to liquefy. Presumably that part of the vulnerable deposit closest to the excitation would be fluidized first. Then the question arises as to how and why would liquefaction trespass beyond that boundary. Surely it couldn't.

The complementary laboratory testing, which involves cyclic loading, I find equally difficult to accept inasmuch as it bears on liquefaction. Apart from believing that such testing would have application only in the case of shear wave transmission, the idea of subjecting saturated sand inside a sealed membrane to as many as a 1,000 stress reversals has always struck me as some kind of abuse of specimen: For some reason or other it makes me think of those bad days in medieval times when confessions were extracted by torture.

As I visualize it, stress reversals result in grain asperities been broken off. These small pieces/dust are not large enough to remain part of the soil-structure. As a result the specimen gradation tends to become gap-graded.

After grain damage progresses and the resisting soil-structure's volume becomes effectively smaller, I think it inevitable that the inside of the membrane becomes increasingly more in contact with specimen water that with specimen solids. I should stress here that what I mean by specimen solids are only those solids comprising the soil-structure, and not including the dust from broken asperities. As the balance of water forces across the membrane depends on the summation of the membrane areas resting on either phase, it seems obvious that the opportunity exists for water pressure inside the membrane to escalate. And this coming about without the need to invoke soil-structure weakening, but rather because of reduced effective lateral confinement. To this we might perhaps add the possible membrane ballooning because of inside temperatures rising from energy expended (work done) by the deviator load repetitions.

I realize this is heretical thinking, but I gained some confidence in this position by the encouragement offered when, towards the end of 1998, Ralph B. Peck wrote me saying: "I share your feeling that much of what we think we know about liquefaction is an artifact of our tests."

Surface Waves and Soil Extension

Just after the Loma Prieta earthquake in October 1989 I went down to California to look at the evidence of damage. Apart from the well reported details, two things struck me as odd and requiring an explanation which didn't seem to fit with the current way of looking at the propagation of energy from the epicenter in the Santa Cruz Mountains to structures in San Francisco about 100 km away.

First was the liquefaction of the dredged sand foundations in the Marina District. As far as I knew attenuation of shear waves were supposed to leave them with little residual energy once they had travelled about 30 km, so how could such destruction be wrought 96 km away? The answer is I believe that it wasn't the shear waves that caused liquefaction, it was the Rayleigh component of a surface wave that did it. Rayleigh waves can travel great distances - they are the geotechnical equivalent of a tsunami. Also, this surface wave, because it causes the ground level to be temporarily super-elevated as it passes would result in elongation of the soil column (ground profile) thus producing a stress path similar to the one replicated by the Vaid triaxial extension work. And as we saw above, soil extension is a most effective way of precipitating liquefaction.

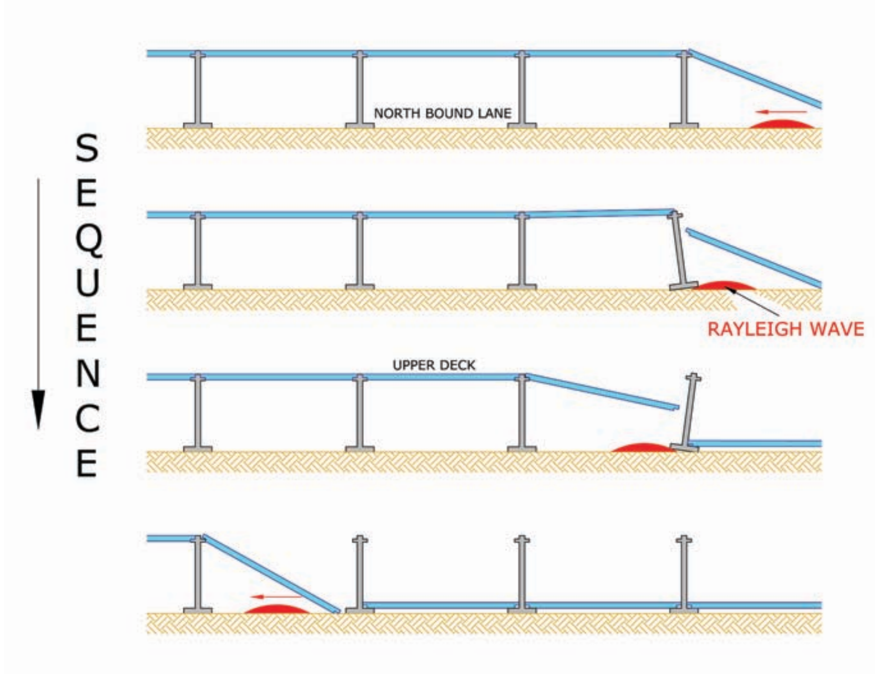

Figure 12. Speculative failure mode for I-880 upper deck.

The second situation which made me wonder was what might have gone on in the ground under the Cypress section of the Nimitz Freeway (I-880) to make the upper deck fall as it did. In his geological narrative *Assembling California*, John McPhee describes it thus:

> "The under road is northbound, and so is disaster. One after the last, the slabs of the upper roadway are falling . . . A man in another car guns his engine, keeps his foot to the floor, and races the slabs that are successively falling behind him."

It is very tempting for me to believe this lucky individual was racing the surface wave radiating out from Loma Prieta. The orientation of the freeway is consistent with this idea, so in Figure 12 I've drawn a cartoon of a possible mechanism which seems consistent with the above evidence. The red bump moving to the left (towards San Francisco) represents the surface deformation due to the seismic wave. Unfortunately, I find the math-physics necessary to calculate the speed of the Rayleigh wave in this particular soil column too intimidating to attempt, and therefore can't say if a car could stay ahead of the wave for a while in this surficial geology.

The point I'm trying to make is this: It is far easier to explain these events by looking at the surface wave and soil-structure extension rather than by examining the damage as if it were caused by shear waves and cyclic loading.

in the Next Article

It is now time to move beyond single particles and approach more closely the practical goal of this series, and that is the generation of pore water pressure in real soils. The next necessary move is to account for the magnification effect due to the crowding of particles inherent in an aggregation of grains packed closely together in a mass. So, in the next article I will develop what I call the K-factor.

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Don't Rely On My Advice!: A Practical Guide to Disclaimers

Neil Abbott

Are disclaimers worth the paper they're written on, assuming they are written at all?

Introduction

One definition of a professional is that they are someone who is paid to give advice. Years of schooling, mountains of experience and natural insight are the necessary ingredients required to render a valuable opinion. Every masterpiece however must have an imperfection; a hallmark of its human creator. It has been my experience that an imperfection gets magnified in reverse proportion to its size; in other words, the smaller the imperfection, the greater its impact when the opinion comes into play. This rule may be because obvious and large imperfections in an opinion are often quickly noticed when the opinion is immediately acted upon, which affords the professional time to re-evaluate and issue a fresh opinion. The minor imperfections such as the failure to take an accurate measurement can often be overlooked in the delivery of a report but can have a great effect upon completion of the project.

Therefore since no opinion is perfect, it is imperfect practice not to issue a disclaimer with your opinion. Disclaimers are ethical, appropriate, acceptable, and all too often constitute overlooked boilerplate. Having a stale-dated disclaimer is sometimes worse than having no disclaimer at all. As will be discussed below contractual provisions that are ambiguous will be read against the party who drafted it.

In this presentation I will discuss the purpose of disclaimers and the overarching principle of disclaimer interpretation contra proferentum. I will then provide a "how to" for drafting an enforceable disclaimer clause and will provide examples of how certain disclaimer clauses have been interpreted by various courts.

The Purpose of Disclaimers

A disclaimer is meant to delineate the scope of rights and obligations stemming from an opinion such as rendered in a report. The question of what to disclaim varies depending on the purpose of the report, but the most common disclaimer is to limit the scope of the report to the site conditions on the day of the inspection, and make no guarantees as to the future condition of what is inspected. For litigation reports the most common disclaimer is to limit the use of the report to counsel and/or the party who has retained the expert for court use only, not to be relied upon for a future project or other party in the litigation or the public at large.

Contra Proferentum

Whether the court will uphold a disclaimer is just as much a question of construction and conduct as what is being disclaimed. The doctrine of *contra proferentum* is applied in the case of disclaimers. In *Bauer v. Bank of Montreal* (1980), McIntyre J., on behalf of the Supreme Court of Canada, stated:

> In construing such a clause, the Court shall see that the clause is expressed clearly and that it is limited in its effect to the narrow meaning of the words employed and it must clearly cover the exact circumstances which have arisen in order to afford protection to the party claiming benefit. It is generally to be construed against the party benefiting from the ex-emption and this is particularly true where the clause is found in a standard printed form of contract, frequently termed a contract of adhesion, which is presented by one party to the other as the basis of their transaction.[1]

How to Draft an Enforceable Disclaimer Clause

1. **The onus is on the professional to bring the disclaimer(s) to the attention of the signing party.**

> The applicability of an exclusion or limitation clause can be challenged on the ground that the party seeking its protection did not bring its existence and inclusion in the contract sufficiently to the notice of the other party at the time of, or prior to the making of the contract, with the result that the latter cannot be taken to have assented to the clause. If this is so, then the clause will not be effectuated …[2]

In Trigg v. MI Movers International Transport Services Ltd., the Ontario Court of Appeal held that the onus on the party seeking to enforce the limitation clause, is greater where a standard form contract is used.[3]

In various sample service agreements and inspection agreements, many disclaimer clauses have disclaimers in caps and others not. This may cause confusion, leading a client to assume that all the disclaimers are in caps. Given the way in which contra proferentum is applied in these cases, a court may find that these disclaimers hidden within the agreement and not in caps should not be upheld. However, in Salgado v. Tooth, many of the provisions of the contract containing disclaimers were upheld while not in caps, while the provision in caps and bolded

[1] Bauer v. Bank of Montreal, [1980] S.C.J. No. 46.

[2] G.H.L. Fridman, Law of Contracts in Canada, 2nd. ed. (Toronto:Carswell, 1986) at para.537.

[3] *Trigg v. MI Movers International Transport Services Ltd.*, [1991] O.J. No. 1548 (C.A.)

was upheld, but its scope was narrowed significantly (see "What to Disclaim - An Example: Salgado v. Toth" below).[4]

2. **Bring the disclaimer(s) to the attention of the signor before the inspection is done.**

   The time when the notice is alleged to have been given is of great importance. No excluding or limiting term will avail the party seeking its protection unless it has been brought adequately to the attention of the other party before the contract is made. A belated notice is valueless.[5]

In Fraser v. Knox, an inspection report was given to the homeowner after the inspection was complete, though the report stated "I hereby authorize the inspection of the Property having read and understood this [Inspection Agreement contained within the Report].[6]" The inspection agreement contained a limitation of liability clause. The court held that the clause is unenforceable since the homeowner should have had the opportunity to negotiate in regards to the term or have the option of retaining an inspector who would not have such a clause in their contract.

3. **Be careful in drafting the disclaimer as it will be strictly construed**.

In *Queen v. Cognos Inc.*, Iacobucci J. states:

It is trite law that, in determining whether or not a limitation (or exclusion) of liability clause protects a defendant in a particular situation, the first step is to interpret the clause to see if it applies to the tort or breach of contract complained of. If the clause is wide enough to cover, for example, the defendant's negligence, then it may operate to limit effectively the defendant's liability for the breach of a common law duty of care, subject to any overriding considerations.[7]

4. **Be precise, complete and comprehensive and read the case of *SALGADO V. TOTH*[8]**

Salgado is instructive on how Courts will interpret contract disclaimers that are not comprehensive or complete. The following contractual provisions were not upheld by the British Columbia Supreme Court:

> 1. The INSPECTOR will perform a VISUAL INSPECTION of the readily accessible and visible areas of the major systems and components of the Primary Residence on the Property and certain built-in equipment and improvements. The inspection and report are not intended to reflect on the market value of the Property nor to make any recommendation as to the advisability of purchase.

The BC Supreme Court held that paragraph 1 of the contract did not contain wording which would limit liability and while the inspector may not have intended the inspection to constitute a recommendation as to the advisability of the purchase, the owner was entitled to rely on such recommendations if made.

> 9. THE INSPECTION AND REPORT ARE NOT INTENDED NOR ARE TO BE USED AS A GUARANTEE OR WARRANTY, EXPRESSED OR IMPLIED, REGARDING THE FUTURE ADEQUACY, PERFORMANCE OR CONDITION OF ANY INSPECTED STRUCTURE, ITEM OR SYSTEM. THE INSPECTOR IS NOT AN INSURER OF ANY INSPECTED CONDITIONS.

The court applied the doctrine of contra proferentum and held that the disclaimer is not broad enough to include guarantees or warranties regarding the present adequacy of the inspected structure.

> 13. It is understood and agreed that should the INSPECTOR be found liable for any loss or damages resulting from a failure to perform any obligations, including but not limited to negligence, breach of contract, or otherwise, then the liability of the INSPECTOR shall be limited to a sum equal to the amount of the fee paid by the CLIENT for the Inspection and Report.

In the contract, "Inspector" was defined as the inspection company and not the inspector personally. Therefore, the court held that this paragraph did not exclude liability for the inspector.

5. **Beware of oral statements made during the inspection.**

In Whighton v. Integrity Inspections Inc., the Inspection Order Agreement contained a limitation of liability clause preventing the client from claiming damages over $10,000:

> 3. LIABILITY. The inspection should not be considered a technically exhaustive inspection or an insurance policy against unexpected house repair/replacement needs. The Client acknowledges that there is risk involved in purchasing a property and that the purpose of the Inspection and the Guarantee is to reduce that risk but not eliminate it. Furthermore, the Client agrees that the performance of the Inspection does not transfer that risk to the Company beyond the Guarantee limits.
>
> .....
>
> **The Company's liability for any Client claims, beyond the Guarantee, is limited to a maximum of the home inspection fee paid. The limitations in liability herein apply to all claims, whatsoever their nature and whether arising from negligence or other tort, in contract or from any other source or cause.**[9]

[4] *Salgado v. Toth*, 2009 Carswell BC 3020 [*Salgado*].

[5] Fifoot and Furmston The Law of Contract, 11th ed. by M.P. Furmston (London: Butterworths, 1986) at 152.

[6] *Fraser v. Knox*, [1998] O.J. No. 4379 at paras. 44-47.

[7] *Queen v. Cognos Inc*., [1993] 1 S.C.R. 87 (S.C.C.) at para. 91.

[8] *Salgado, supra* note 4 at para. 13.

[9] *Whighton v. Integrity Inspections Inc.*, 2007 CarswellAlta 376 at para. 47

The issue in this case was whether the clause is broad enough to include gratuitous oral statements, including statements that the home was a "great house" in "good shape" and that necessary repairs would be $6,000.[10] The Alberta Court of Queen's Bench held that as the statements were made outside the terms of contract, the statements were not protected by the limitation of liability clause:

> The clause in this case purports to exclude liability beyond the Guarantee for all claims "whatsoever their nature and whether arising from negligence or other tort, in contract or from any other source or cause." Strictly construed against Housemaster, this clause should be read narrowly to exclude liability for a breach of contract or negligence in relation to the performance of that contract. Without clearer construction, the clause cannot exclude Housemaster from any negligence under any circumstances. Therefore, the clause cannot protect Housemaster from liability for negligence in relation to actions performed outside the terms of the contract.[11]

The contract did not provide for assessments of repair costs and it was not in the inspector's practice to provide the assessment, so such a representation was made outside the terms of the contract.

Note that the court's finding was assisted by a clause in the agreement related to oral representations, stating that the written report constituted the inspection results and that oral representations would not alter the interpretation of the inspection results.

6. Incorporate all documents into the Contract or Agreement containing the disclaimer(s).

In *Salgado v. Toth*, clause 16(b) stated "[B]y signing the Property Inspection Contract, the CLIENT acknowledges, covenants and agrees that: b) The INSPECTOR has not made any representations or warranties other than those contained in the Contract." Clause 16(b) was not enforced by the court as the Inspection Report was a separate document and the representations and warranties were contained in that report, not the contract. The Contract did not incorporate the subsequent reporting pages on which the representations and warranties were contained. The court held that

> While it may have been the intent of paragraph 16(b) to exclude representations or warranties that arose outside the Contract, it could not have been in the contemplation of the parties that a reference to a document containing no representations or warranties would exclude representations or warranties that were made to induce the Plaintiffs to enter into the Contract or which were contained in the oral or written report subsequently provided by Mr. Toth.[12]

Other Examples of Disclaimers

The following contractual provisions were upheld by the British Columbia Supreme Court in *Salgado v. Toth:*

> 2. The condition of certain systems, components and equipment will be randomly sampled by the inspector. Examples of such systems, components and equipment are window/door operation and hardware, electrical receptacles, switches and lights, cabinet/countertop mounts and functions, insulation depth, mortar, masonry, paint and caulking integrity and roof covering materials. Furniture, rugs, appliances, stored items, etc. will not be moved for the inspection.
>
> 3. The INSPECTOR will give a professional opinion on whether those items inspected are performing their intended function at the time of the inspection or are in need of immediate repair. The inspection and report are based upon observations of conditions that exist at the time the inspection was performed.
>
> 4. Cost estimates, if provided, are "ballpark" estimates only and are not intended to be relied upon by any person for accuracy. The CLIENT should obtain written bids from qualified licensed contractors in order to determine the possible cost of repairs.
>
> 6. The Client is encouraged to participate in the visual inspection process and accepts responsibility for the consequences of electing not to do so, i.e. incomplete information being available to the Inspector. This Client's participation shall be at the Client's own risk for injuries, falls, property damage, etc;[13]

Conclusion

Disclaimer clauses are a professionals' shield to defend themselves against the client's sword. Disclaimers have become a necessary part of doing business in the litigation environment. They are ethical and mandatory. Your disclaimer should be read and updated and not casually inserted as part of the boilerplate. Your disclaimer may be negotiated, limited or expanded depending on the circumstances but it should always be considered as your safeguard to ensure that your professional opinion is not inappropriately used…but don't rely on my advice!

Neil Abbott, Partner, Gowling Lafleur Henderson LLP, 1 First Canadian Place, 100 King Street West, Suite 1600, Toronto, Ontario M5X 1G5, T 416 862-4376, F 416-863-3476, neil.abbott@gowlings.com

Bauer v. Bank of Montreal, [1980] S.C.J. No. 46.

10 *Ibid.* at para. 30.

11 *Ibid.* at para. 51..

12 *Salgado, supra* note 4 at 77.

13 *Ibid.* at 13.

University of Virginia Engineering Professors Provide Valuable Resource to Calculate Pharmaceuticals Present in Virginia Wastewater

In response to media headlines about the presence of pharmaceuticals in drinking water, professors from the University of Virginia School of Engineering and Applied Science have developed a website, "V-PharmaCalc," that allows Virginians to view estimated concentrations of various prescription and generic drugs present in their local wastewater - before it is treated and released to the environment.

"It is important for Virginia residents to use our website as a tool to understand the type and amount of drugs that are present in their local wastewater," says Lisa Colosi, assistant professor of civil and environmental engineering, who co-led the website development project. "This understanding will allow Virginia residents to put information they gather from the media into perspective.

"While most scientists agree that the presence of pharmaceuticals in the water supply is undesirable, the concentrations of these drugs are very low and it is currently unknown what direct effects they may have on human health."

Colosi worked on the project with colleague James Smith and doctoral student Karl Ottmar, both from the Department of Civil and Environmental Engineering. Several undergraduates also worked on the project, including rising third-year student Matthew Quinn.

In addition to providing estimates of the concentrations of pharmaceuticals in local wastewater, the website also features additional educational resources. The website explains how pharmaceuticals enter wastewater, typically when they are flushed down the toilet or poured into a garbage disposal. (As an alternative, the researchers recommend combining the drugs with coffee, kitty litter or another unappealing substance and disposing of them in the trash.)

The site also briefly explains standard wastewater treatment processes in the United States and Europe and how pharmaceuticals are affected by the treatment process.

"We hope that the resources featured within the website will de-sensationalize the issue of pharmaceuticals in wastewater," Colosi said. "Additionally, we hope to provide individuals with an understanding of how their personal choices directly affect the water quality and the ecosystem within Virginia and on a global scale. We hope that this will encourage individuals to consciously make decisions that are better for the water quality and health of the ecosystem."

The Virginia Environmental Endowment sponsored the project as part of its mission to improve the quality of the environment by preventing pollution, conserving natural resources and promoting environmental literacy.

"Dr. Colosi and her colleagues at the University of Virginia have clearly demonstrated a way for citizens to find out what kind of pharmaceuticals are in Virginia's water," said Gerald P. McCarthy, executive director of the Virginia Environmental Endowment. "They have also offered constructive ideas for what to do about that. The Virginia Environmental Endowment is very pleased to have supported such useful research as well as the related graduate and undergraduate education obtained by the students who worked on this important project."

BOOK REVIEWS

The Story of the Vaiont
By Eduardo Semenza (1927 – 2002)

Told by the geologist who discovered the slide

Published in Italian in 2001
English edition published in 2010

On 12 February 2008, while launching the International Year of Planet Earth, UNESCO cited the Vaiont Dam tragedy as one of five "cautionary tales", caused by "the failure of engineers and geologists (Wikipedia).

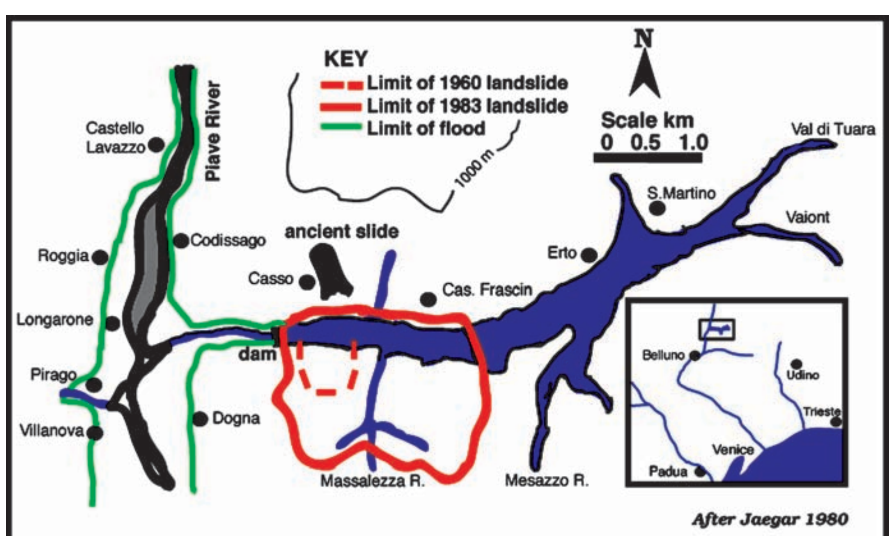
The Vaiont concrete arch dam was constructed in 1960 near Longarone in northeastern Italy, about 100 km north of Venice. The dam and reservoir were to form part of a hydropower project to provide power to Italy's' northern cities

The Vaiont slide occurred in October 1963 during the third filling of the Vaiont reservoir. The axis of the slide mass was less than 1 km upstream of the 300 m high concrete arch dam. The water displaced by the slide overtopped the dam, which caused considerable damage downstream and the death of almost 2000 persons. The maximum height of the wave was about 210 m, above the top of the concrete arch dam.

Initial engineering studies for the Vaiont dam started in 1926 under the direction of Carlo Semenza (father of the author) and continued up to 1940. World War II intervened but studies recommenced in 1948 and excavations for the foundation of the concrete arch dam started in 1957. During these periods site investigations and studies concentrated on the permeability and stability of the abutments for the dam. Studies of slope stability within the reservoir were not made. Up to that time, it was not general practice to request geological studies for projects.

In March 1959 the Pontesi landslide occurred near Fagare, about 80 km south of Vaiont dam site. This prompted the need to verify the possible risks of landslides in the Vaiont reservoir. Edoardo Semenza, a recent geology graduate was commissioned in July 1959 to make the study. By the end of August 1959 he verbally communicated his findings concerning slope stability to the project managers. He notes that there are sites of various paleoslides, only one of which is potentially dangerous – the "Colle Isolato" on the right side of the valley. Edoardo Semenza hypothesizes that new movements of the slide mass could be produced by filling the reservoir. The story of Vaiont begins.

Alfred J Hendron and Franklin D Patton in their preface to the English version note: "In some ways the story can be compared to a Greek tragedy. The project engineer, Carlo Semenza, was told at various times by one or more of his experienced consultants (Dal Piaz, Caloi, Penta and Muller) and others that a it was unlikely: 1), there was significant previous landslide, 2), the moving rock could be stabilized by drainage, and 3) further movements of the slide would be fatal to the project. On the other hand his son, Edoardo, a recent geology graduate, was telling him: 1), there was a very large pre-existing slide just upstream from the dam, 2) the slide had previously moved across the valley, 3) the slide was resting on weak materials, and 4) the old slide could be reactivated by rising the reservoir. As we now know Edoardo was correct. We think that these conflicting technical opinions provided the real drama prior to the slide, not the contrived plots of the reporters and authors of the play and movie".



Hendron and Patton note: “The period from 1959 to 1985 when we completed our report (The Vaiont Slide, a geotechnical analysis based on new geological observations of the failure surface, Technical report GL-85-5, Department of Army, US Army Corps of Engineers, Washington D.C. 2 Vols), was one of significant changes in the fields of geotechnical engineering, rock mechanics and hydrogeology. In particular the tools for investigation, testing and analysis of landslides were changing rapidly”.

The saga of events during filling of the reservoir, which included the transfer of ownership from SADE to ENEL, death of the Carlo Semenza (October 1961) and Dal Piaz as a result of an automobile accident (April 1962), emphasize the challenges the management of large projects, the importance of communications, detailed recording of events and probably “project politics”.

The index to this book is on the last two pages. I found it useful to read both the index and Appendix 2, “Summary Succession of Events with Special Attention to Geological Considerations” before working my way through the story. (To me this is akin to reading the programme notes for the Opera before it begins – I do not understand the language, but I can follow the story).

The Vaiont Slide in 1963 is the classic case in recognizing the need in advance, by regulators, designers and owners, the geological difficulties in constructing and managing new water reservoirs and very large mine-waste management storage systems.

The Vaiont Story case should be included in all courses of geotechnical/geological engineering and related fields.

John Gadsby – July 2011

Rudolph Glossop and the Rise of Geotechnology

Most North American geotechnical engineers will ask “who was Rudolph Glossop and what did he do for geotechnology?”

My personal contact with Rudolph Glossop was in 1949/50, midway in the last century, when I was employed as an “engineer in training” in Soil Mechanics Ltd. in London U.K. Soil Mechanics was a subsidiary to the prestigious UK contractor Mowlem. The directors of Soil Mechanics were: H.J.B. Harding, H.Q. Golder and R. Glossop. I was assigned to be Golder’s E in T and had little contact with Harding and Glossop, apart from taking them a cup of tea at exactly 10.00 am when their E in T was away.

Glossop’s entire career (1924 to 1961) was essentially with contracting aspects of geotechnology.

This is an absolutely fascinating book on the early history of geotechnology, from 1922 to 1961, as seen through the written words of Rudolp Glossop and his contributions to the growth of geotechnics. The book is compiled from selected extracts from Glossop's journals, diaries and letters and includes correspondence with Bjerrum, Skempton and Terzaghi. It starts in 1922, when Glossop did summer field work at the old Levant mine near Land's End, through to 1961 when his diaries cover issues associated with the construction of the Derwent Dam in N.E. England.

Glossop graduated as a mining engineer from the Royal School of Mines in 1924. In his early years he worked with mining companies in Canada (1924-1930) and the Gold Coast (1933-37) and a short introduction to civil engineering working on the construction of the Leicester Square underground station in (1930-32). Glossop joined Mowlem in 1937 to manage their partnership with Siemens. It is a joy to read his words, which often cover the contractual and people issues associated with engineering works. Skempton, in his obituary of Glossop, refers to: *works of original scholarship, written with the clarity and style of which Glossop was a master - by his own example and by the example of others, he never lost sight of the importance of bringing together the practical and academic aspects of both geology and soil mechanics"*.

Some examples: Glossop describes his summer assignment at the Levant Mine in 1922 near Land's End: "*but if the place was beautiful, working conditions, both on surface and underground, were primitive in the extreme, for mining at Levant started in 1820 and in 1922 machinery and methods had not changed very much"*. His description of using wooden ladders to climb down and up about 1000 feet to the mine workings are captivating and could be well used in the setting to a mystery novel. In 1931 he writes about the challenges of finding work in London following the financial crash,

What a great read - highly recommended for all engineers – regardless of where or for whom they work.

which curtailed his dreams of completing a doctorate at Harvard. Glossop describes his interview with the general foreman of Brand's, another UK construction company. He was offered a job for 3 GB pounds a week to work as a miner's labourer. *For a young English engineer to have accepted such an offer would, in those days, have been unthinkable, but after my years in Canada I was free from such snobbery"*.

Flash forward to 1961. At this time, Glossop is a director of Mowlems (Soil Mechanics Ltd). There are difficulties with the construction of the Derwent Dam in N.E. England. Boreholes put down by Soil Mechanics Ltd at the start of the contract showed a geological section markedly different from that communicated in the contract documents. This was brought to the attention of the Engineer and the significance with respect to ground water lowering. The construction programme envisaged by the Engineer could not be carried out and work was disrupted for over a year. The client had retained Peter Rowe to carry out a detailed analysis of the pumping tests. Glossop writes: "*Rowe's report purports to demonstrate that all pumping tests show that Soil Mechanics Ltd's view of the geology of the site are irrelevant, and that as regards response to pumping from wells, the groundwater behaves as might be expected from the study shown on the contract drawings"*. Soil Mechanics Ltd. prepared their report with the assistance of Skempton. Glossop describes the meeting in September 1961, at which Rowe and Skempton were present to discuss the implications of these reports. He writes; "*although I believe we somewhat weakened Rowe's position, the fact remains that our views were not wholly accepted. If their views prevail, the consequences to us will be grave indeed. To sum up, the post-contractual causes of our troubles are, in my opinion due to:*

1. *The fact that the Engineer will not accept that his original site investigation was badly done, and that the whole work has been disrupted, largely at our expense, for a year.*
2. *We have followed a policy of appeasement and failed to press home our advantage a year ago by invoking Clause 12(2). Our position is now no stronger than it was then and it is harder for the Engineer to retreat, for about 250 000 GP pounds has been frittered away in the meantime.*
3. *We may have lost the confidence of the Engineer by not accepting his invitation to collaborate with him.*

Glossop continues by recommending future action by John Mowlem & Co. Ltd. and measures to prevent a reoccurrence of such a situation.

The book is only 280 pages, which includes selected writings and letters. The editor, Ronald E. Williams, has carefully selected Glossop's words. In today's world of computers and instant communications, and of so called "standard project reporting systems", Glossop's writings are a classic example on what is required in preparing project diaries and as-built reports. I was fortunate to receive a copy of this book from John Dunnicliff – I heartedly thank him. What a great read - highly recommended for all engineers – regardless of where or for whom they work.

John Gadsby, Vancouver, Canada.

Published by: Whittles Publishing; Dunbeath, Scotland. Available on Amazon.ca for about $60.00.

MEMORIAM

Carl Benson Crawford 1923-2010

Carl was born in Dauphin, Man. on Oct. 2, 1923. He died in Vancouver on Aug. 28, 2010, surrounded by his wife, Adah, and their four children, Nora, Henry, Meg and Blair. Carl and Adah were married for nearly 62 years and during that period shared the joys of having children and of travel to many countries, both for pleasure and for Carl's work.

Carl served as a navigator in the Second World War. After the war, Carl attended Queen's University in Kingston graduating in 1949 with a degree in civil engineering, followed by postgraduate degrees from Northwestern University in Illinois and Imperial College in London.

While at Queen's, Carl attended a lecture by R.F. Legget and was so impressed, he joined the National Research Council in Ottawa working for Legget in the Soil Mechanics Section of the Division of Building Research. This launched Carl's illustrious career in geotechnical engineering.

Carl is perhaps best known for his pioneering work on Leda clay, a highly sensitive clay prevalent in the Ottawa area and the cause of numerous landslides and major settlement problems. Carl developed testing apparatus and measurement techniques to measure the behaviour and properties of Leda clay and published several papers on this work. He also worked closely with Laurits Bjerrum and other leading researchers at the Norwegian Geotechnical Institute who were studying the sensitive Scandinavian clays at the same time.

Carl became Director of the Division of Building Research in 1974, a position he held until his retirement in 1985. During this period, he was Chairman of the National Research Council's Associate Committee on Soil Mechanics which had considerable influence on geotechnical research and practice in Canada.

After his retirement, Carl continued his research interests spending time at Cambridge University in England, the Norwegian Geotechnical Institute in Oslo, the Centre for Cold Oceans Research in St. John's, NL, and at the University of British Columbia. During this period, Carl documented several valuable case histories where long term settlement records could be compared with predicted settlements.

Carl received many honours over the course of his career, including the 6th R.F. Legget Award from the Canadian Geotechnical Society in 1975; the Julian C. Smith Medal from the Engineering Institute of Canada in 1989; and the 1996 R.M. Quigley Award for Carl and his co-authors for the best paper of the year in the Canadian Geotechnical Journal. Carl was elected as a Fellow of the Engineering Institute of Canada (FEIC) in 1983 and, in 1985, he was invited by the Canadian Geotechnical Society to undertake a two week Cross Canada Lecture Tour. In 1984, he received an honorary doctorate of law from Concordia University in Montreal.

In addition to his family, one of Carl's true pleasures was the family cottage that he had designed and built at Sharbot Lake, located about two hours southwest of Ottawa. After Carl had retired, and he and Adah moved to Vancouver, every summer they would make the long drive back to the cottage, stopping to visit friends and family along the way. Over the years they made 40 of these trips.

Carl Crawford made a significant impact in the field of geotechnical engineering research, and particularly our knowledge of the properties and behaviour of sensitive clays. His work is an enduring contribution to international geotechnical practice.

MILE STONES

John Mooney Appointed as VP Sales and Marketing

The board of directors of Titan Environmental Containment Ltd are pleased to announce the appointment of Mr. John Mooney as Vice President, Sales and Marketing. The strategic appointment of Mr Mooney will support the growth and success that Titan has experienced as one of Canada's leading geosynthetic contractors and suppliers. Mr Mooney is acknowledged as one of the pioneers of geosynthetic's in Canada and brings to his new position over thirty years of experience in the supply and installation of a wide range of geosynthetic materials. Most recently Mr. Mooney worked as a consultant with a European company helping to introduce their technology to Canada. Previously he held a senior management position with a Canadian geosynthetic contractor. In his new position Mr. Mooney is responsible for all sales and marketing activities in Canada, the United States and the Caribbean. Mr. Mooney's appointment is effective immediately; he is based in Guelph, Ontario.

Titan Environmental Containment Ltd is acknowledged as one of Canada's leading geosynthetic suppliers and contractors with a constant focus on building long term relationships. A leader in the supply and installation of bolted steel tanks, Titan is head quartered in Winnipeg with branches in British Columbia, Ontario and Newfoundland. Titan is committed to preserving the environment for future generations by utilizing only quality materials, installed by experienced personnel using state of the art equipment.

MARKET PLACE

Titan Awarded Contract for Largest Lined Cap in Canada

Titan Environmental Containment Ltd of Winnipeg, Manitoba has been awarded the contract to cap the Lynn Lake mine site. The site is over 800,000m^2 in area and will require the installation of a 1.5 mm textured HDPE geomembrane.

Mr Brett Buckard, principle at Titan commented, "this project has had several redesigns over the past couple of years. We are delighted to have been chosen as the geosynthetc installer for possibly the largest cap in Canada to use a geomembrane as the main cover system. This project will test all our resources as it requires us to meet an extremely tight schedule, coordinating material supply and providing a very large and experienced workforce capable of meeting our client's needs."

The installation of the geomembrane is expected to begin in August and be completed by December.

Titan Environmental Containment Ltd is one of Canada's leading geosynthetic suppliers and installers specializing in the installation of HDPE and PVC geomembranes, geocomposites and Bolted steel tanks. Titan is head quartered in Winnipeg with branch offices in British Columbia, Ontario and Newfoundland.

Pile Dynamics, Inc (PDI) Announces the PIR Viewer

The PIR Viewer is a new accessory for the Pile Installation Recorder (PIR), an Automated Monitoring Equipment system for augercast (CFA) piles.

Pile Installation Recorder is an instrument used for monitoring installation of augered cast-in-place, continuous flight auger and drilled displacement piles. The PIR assists in the correct installation of these piles by displaying target versus actual pumped concrete / grout volume in real time. The equipment is installed on the crane, and easily monitors the installation of every pile on site.

The PIR Viewer is a hand-held wireless device that allows on site personel (inspector, foreman, engineer, etc.) to view what the PIR main unit is displaying to the crane operator. The progress of the drilling and grouting operations are seen in real time on both the main unit and on the PIR Viewer. In addition to receiving data, the PIR Viewer allows the inspector or foreman to enter the observed grout return, which is then recorded in the PIR along with the entire installation record. PIR Viewer features include:

- All PIR results, including incremental grout volumes displayed on screen in real time
- Compact and light weight (135 x 104 x 52mm; 0.45kg)
- Red flag indicator making it easy to identify under-filled pile sections, which can be immediately corrected, saving money and material costs
- High contrast color LCD with LED backlighting, for all lighting conditions
- Displays pump strokes for log
- Automatically receives summary report from PIR Main at pile completion so entire installation can be viewed

Pile Dynamics offers the entire PIR system for sale (or rental for USA domestic customer only). The PIR Viewer is an optional accessory for the latest generation PIR systems. Please contact us at Sales@pile.com for a quote or visit us online at www.pile.com.

ASFE Offers Free Publications

ASFE has added four brownfield-redevelopment publications to its online store. The four publications are:

- The Brownfields Tax Incentive comprises a brownfields tax-incentive fact sheet, guidelines for using the tax incentives, and answers to frequently asked questions, along with case histories and information about other resources.
- Revitalizing America's Mills: A Report on Brownfields Mill Projects relates case histories that chronicle some of the challenges faced and innovative solutions found during the revitalization of more than 350 abandoned and potentially contaminated mill sites.
- Mine-Scarred Lands (MSL) Initiative Tool Kit is designed to help communities clean up and revitalize former mines by sharing models from the six MSL Initiative projects focused on the clean-up and redevelopment of hard-rock- and coal-mine-scarred lands. The document also provides links to a wide range of resources.
- From Landfill to Landmark: Save The Bay Center reviews Save The Bay's seven-year Fields Point development project, documenting the interplay between permitting, financing, and clean-up requirements, Save The Bay's decision-making, and the project outcomes. The Save The Bay Center demonstrates how redevelopment of brownfield properties builds on prior public and private investment, takes advantage of existing infrastructure, and helps revitalize communities.

All four publications are available free at www.asfe.org.

You Never See Our Best Work...
But you have confidence in knowing we've been there.™
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