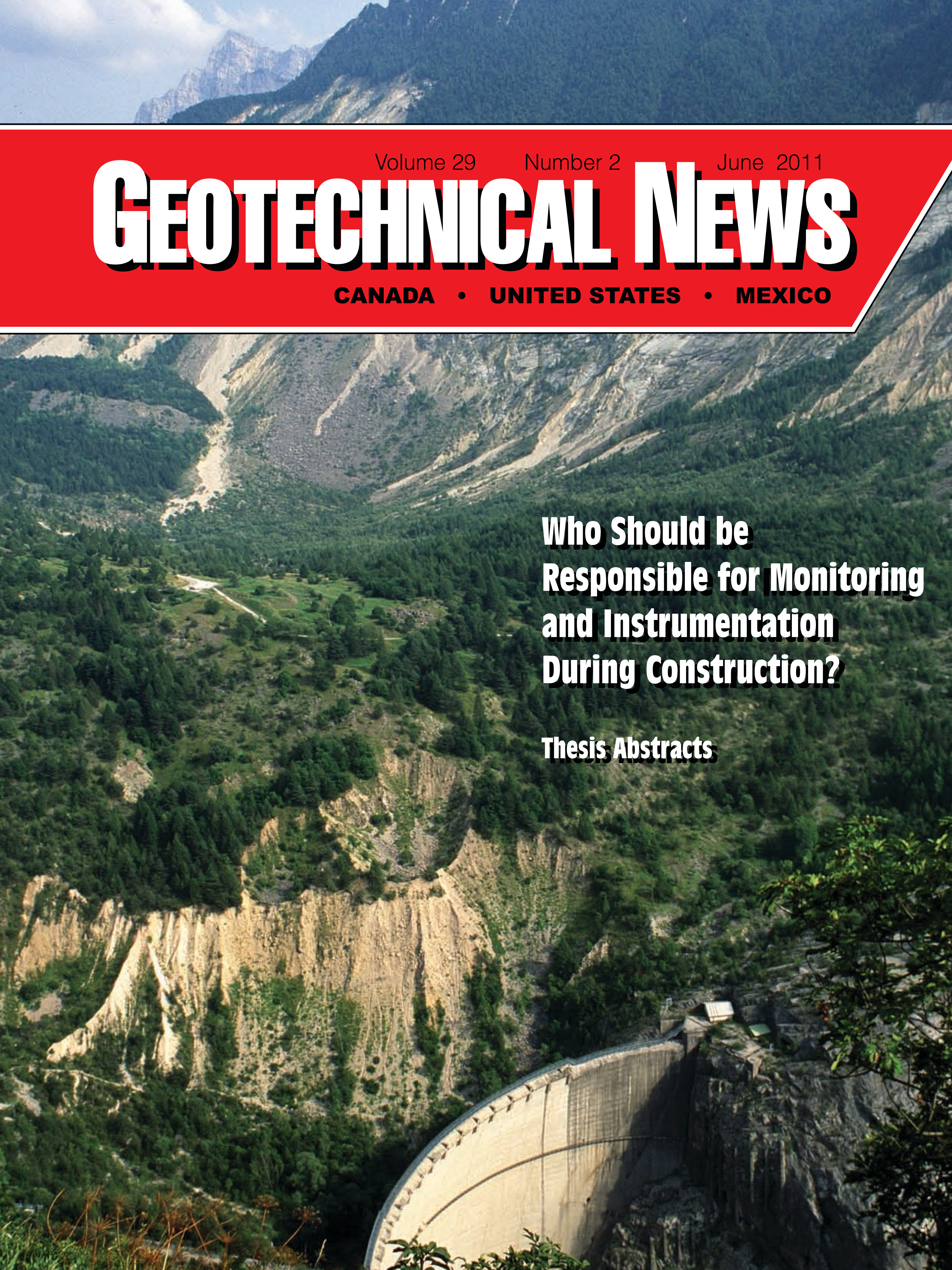
Volume 29 Number 2 June 2011

GEOTECHNICAL NEWS

CANADA • UNITED STATES • MEXICO



Who Should be Responsible for Monitoring and Instrumentation During Construction?

Thesis Abstracts

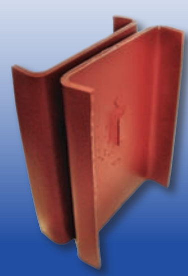
TITUS STEEL

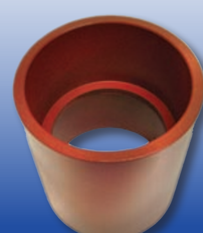
DRIVING POINTS & SPLICERS

INTRODUCING THE REDESIGNED
"HD ROCK INJECTOR"

- CHIPS AND SEAT INTO **SLOPING ROCK** UP TO **50°**
- TESTED TO OVER **700 TONS** STATIC LOAD
- CASE HARDENED STEEL, 3 TIMES THE MASS AND WEIGHT OF ANY OTHER POINT
- **50 YEARS** EXPERIENCE
- THE ONLY **HEAVY DUTY ROCK INJECTOR** IN THE WORLD (H & PIPE)
- **SPECIFY TITUS POINTS...AND BE ASSURED**

Also Available:





Standard "H" Point • Open Cutting Shoe • "H" Splicer • Inside Pipe Splicer • Outside Pipe Splicer

For research reports please call or e-mail

1. Pile Driving to bearing on steeply sloping hard bedrock by Trow Ltd.
2. Driving into sloping rock bed
3. Static load testing

TITUS STEEL CO. 6767 INVADER CRES. MISSISSAUGA, ON. L5T 2B7

1-888-564-7904 • FAX: 905-564-2450 • sales@titussteel.com • www.titussteel.com

GEOTECHNICAL NEWS

Volume 29 Number 2 **June 2011**

Publisher **John W. Gadsby**

Managing Editor **Lynn Pugh**

Graphics **George McLachlan**

Editors

Linda Bayer, Phil Bruch, Robert Chapuis, John Dunnicliff, Paolo Gazzarrini, Saeed Otufat-Shamsi, Ward Wilson

Managing Editors and Advertising NEW
BiTech Publishers Ltd.
103 - 11951 Hammersmith Way
Richmond, British Columbia
Canada V7A 5H9.
Tel: 604-277-4250, Fax: 604-277-8125
email:gn@geotechnicalnews.com
web: www.geotechnicalnews.com

Canadian Editorial Office
Canadian Geotechnical Society
Phil Bruch, Editor, CGS News
email: Phil_Bruch@golder.com

United States Editorial Office
Geo-Institute of the American Society of Civil Engineers
Linda R. Bayer, 1801 Alexander Bell Drive,
Reston, VA 20191-4400
Tel: 703-295-6352 Fax: 703-295-6351
email: lbayer@asce.org

Computing in Geotechnical Engineering
Saeed Otufat-Shamsi, 4188 Hoskins Road,
North Vancouver, BC V7K 2P5,
Tel: 604-603-5650,
email: Saeed@novotechsoftware.com

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

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

The Grout Line
Paolo Gazzarrini, 12-2242 Folkestone Way,
West Vancouver, BC, V7S 2X7
Tel: 604-913-1022, Fax: 604-913-0106
email: paolo@paologaz.com

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

Paper Subscriptions
Geotechnical News is published quarterly. Subscription rates within North America are $51.00 per year; overseas rate is $85.00 US per year through BiTech Publishers Ltd.

eGN
Geotechnical News is also available in an electronic version.
Visit www.geotechnicalnews.com for details.

Membership Information
Canadian Geotechnical Society
Gibson Group Association Management
Wayne Gibson, 8828 Pigott Road
Richmond, BC V7A 2C4
Tel: 604-277-7527
email: cgs@cgs.ca

Printed in Canada.

CONTENTS

VOLUME 29 NUMBER 2 JUNE 2011

Cover

Vaiont Landslide. The book The Story of Vaiont will soon be available from BiTech. (Photo by G. De Biasi).

CANADA

CGS NEWS

Message from the President

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

This is my second message to CGS members as your President. Our new Executive Committee (EC) is now well into their fourth month in their two year term and, at the time of writing, are preparing for the first EC meeting in Calgary on April 9, 2011. During this initial warming up period we heavily rely on our Secretary General, Dr. Victor Sowa, and our Administrator, Wayne Gibson. You will have now seen our new website which is largely due to the efforts of Wayne Gibson and our past VP, Communications, Ms. Stephanie Perrett. We hope that you find it more effective than the previous website. If you have comments, please contact our VP, Communications, Dr. Jean-Marie Konrad.

As part of our responsibilities on the CGS EC, we also represent the CGS on other Canadian technical associations to further communications among such associations. Our VP, Technical, Dr. John Sobkowicz, represents us on the council for the Canadian Foundation of Earth Sciences (CFES). Ms. Marcia MacLellan, our Local Sections Representative, will represent us on the Board of the Canadian Society of Civil Engineers. The umbrella organization for engineering societies in Canada is the Engineering Institute of Canada (EIC) which will have its 125th anniversary this year. I represent the CGS on the EIC council.

I had the pleasure of attending my first EIC Awards dinner as President in Ottawa on March 5, 2011. I sat at a table with Dr. Liam Finn who won the K.Y. Lo Medal. This was especially gratifying because Dr. K.Y. Lo and his son were also at the same table. Our Dr. Wayne Savigny of BGC Engineering and Dr. Jean Hutchinson of Queen's University were elected as Fellows of the EIC. A distinguished Canadian geotechnical engineer, Dr. Graham Morgan, was also elected as a Fellow of the EIC through his affiliation with the Canadian Society of Senior Engineers. I was interested to learn that Dr. Morgan started his career with Geocon in Ontario in the 1950s as did so many other geotechnical engineers of that era. Our

1986 Leggett Award winner, Mr. Fred Matich, started Geocon in the 1950s.

The EIC awards are among the most prestigious engineering honours in Canada. The CGS actively promotes its members for EIC awards through its participation in the EIC. The IEEE Canada, also a member of the EIC, has over 15,000 members while the CGS has 1,200 members. Although much smaller than many other constituent societies of the EIC, the CGS has represented its members well over the decades. Do not forget to nominate your deserving colleagues for these EIC awards!!

Coming back from the EIC awards dinner, I became interested in how many Canadian geotechnical engineers had received the Order of Canada, our country's most prestigious service award. Among those who have received the Order of Canada in no particular order are: Dr. Robert Leggett (d.), Dr. G.G. Meyerhoff (d.), Dr. Robert Hardy (d.), Dr. Jack Clark (d.), Dr. Del Fredlund, Dr. Norbert Morgenstern, Dr. Jack Mollard, and Mr. Hector Jacques. That so many geotechnical engineers have won this distinguished award is emblematic of our proud CGS traditions and the important role of geotechnical engineering in Canada.

The CGS will be sponsoring three important conferences in 2011. At the time of publication, the first conference, 5th Canadian Conference on Geotechnique and Natural Hazards will already have happened in Kelowna. The next is the Slope Stability Conference in September in Vancouver. A university colleague of mine, Dr. John Simmons who did his PhD at the University of Alberta in the 1970s, will be making the trek from Queensland to present a paper. This underscores the dual purpose of our conferences; dissemination of the latest technical advances and the renewing friendships and technical collaboration. Following that will be the PanAm-CGS Conference in Toronto, perhaps the most important conference sponsored by the CGS in the last couple of decades. Our organizing committee is working hard and expects a successful Conference. Even so, this conference will be an opportunity to showcase Canadian capabilities in geotechnique. If you are choosing among international conferences to attend this year, this should be your choice.

Come out to your local section's presentations to learn about new technology and exchange ideas. I hope to see CGS members in large numbers at the PanAm-CGS Conference and trust that you are finding our new website as effective as we hope it to be.

Le Message du président

Ceci est mon deuxième message aux membres du CGS en tant que votre président. Notre nouveau Comite Exécutif (CE) est désormais bien dans leur quatrième mois de leur mandat de deux ans et, au moment de l'écriture, se préparent pour la première réunion CE a Calgary le 9 Avril, 2011. Au cours de cette période de réchauffement initial, nous comptons beaucoup sur notre Secrétaire General, Dr. Victor Sowa, et notre Administrateur, Wayne Gibson.

Vous aurez donc vu notre nouveau site web, qui est en grande partie grace aux efforts de Wayne Gibson et notre ancien VP, Communications, Mme Stephanie Perrett. Nous espérons que vous trouverez qu'il est plus efficace que le site précédent. Si vous avez des commentaires, s'il vous plaît communiqué avec notre vice-président, Communications, Dr. Jean-Marie Konrad.

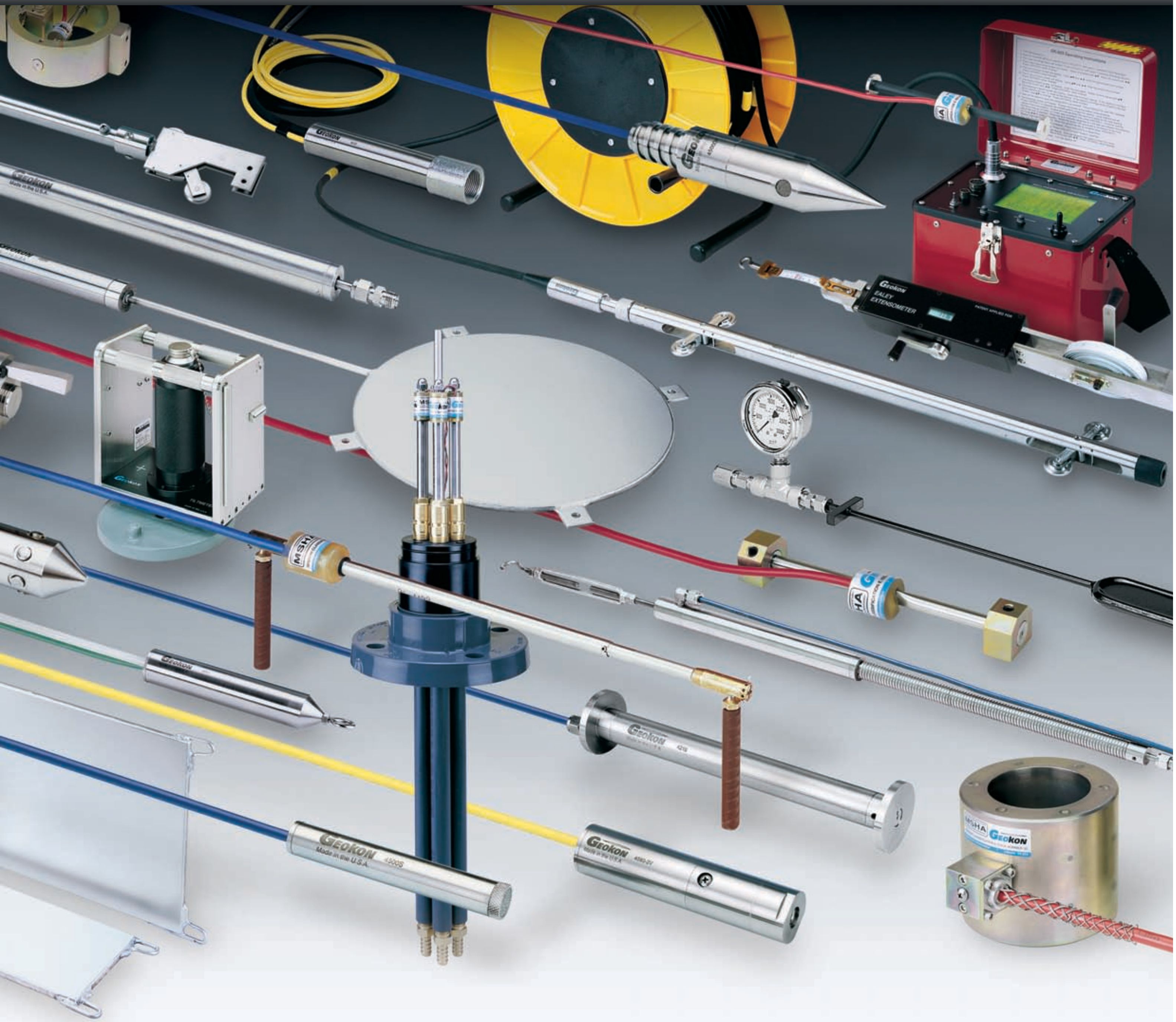
Dans le cadre de nos responsabilités sur le CE CGS, nous représentons également le CGS sur d'autres associations canadiennes techniques à de nouvelles communications entre ces associations. Notre VP, Technique, Dr. John Sobkowicz, nous représente au sein du conseil de la Fédération canadienne des Sciences de la terre. Mme Marcia MacLellan, notre Représentante des Sections Locales, va nous représenter au conseil d'administration de la Société Canadienne d'Ingénieurs Civils. L'organisation faîtière des sociétés d'ingénierie au Canada est l'Institut Canadienne des Ingénieurs (ICI) qui aura son 125e anniversaire cette année. Je représente la CGS au sein du conseil EIC.

J'ai eu le plaisir d'assister à mon premier Gala des prix du CPN en tant que président à Ottawa, le March5, 2011. Je me suis assis à une table avec le Dr Liam Finn, qui a remporté la K.Y. Lo médaille. Ce fut particulièrement gratifiant parce que le Dr K.Y. Lo et son fils ont été également à la même table. Notre Dr Wayne Savigny de BGC Engineering et le Dr Jean Hutchinson de l'Université Queen's ont été élus Fellows de l'ICI. Un éminent ingénieur géotechnique Canadien, Dr. Graham Morgan, a également été élu Fellow de l'EIC à travers son affiliation avec la Canadian Society of Senior Engineers. J'ai été intéressé d'apprendre que le Dr Morgan a commencé sa carrière avec Geocon en Ontario dans les années 1950 comme beaucoup d'autres ingénieurs en géotechnique de l'époque. Notre lauréat du Prix 1986 Leggett, M. Fred Matich, a commencé Geocon dans les années 1950.

Les prix EIC sont parmi les honneurs d'ingénierie les plus prestigieux au Canada. Le CGS promeut activement ses membres pour les prix EIC grâce à sa participation à l'EIC. L'IEEE Canada, également membre de l'EIC, a plus de 15,000 membres alors que la CGS a 1,200 membres. Bien que beaucoup plus petits que beaucoup de sociétés d'autres constituants de l'EIC, la CGS a représenté ses membres ainsi au fil des décennies. N'oubliez pas de nommer vos collègues méritants pour ces prix EIC!

En revenant du EIC Gala des prix, je me suis intéressé à combien ingénieurs en géotechnique canadienne avait reçu l'Ordre du Canada, le prix de service le plus prestigieux de notre pays. Parmi ceux qui ont reçu l'Ordre du Canada en aucun ordre particulier sont : Dr. Robert Leggett (d.), Dr. G.G. Meyerhoff (d.), Dr. Robert Hardy (d.), Dr. Jack Clark (d.), Dr. Del Fredlund, Dr. Norbert Morgenstern, Dr. Jack Mollard et M. Hector Jacques. Que tant d'ingénieurs géotechniques ont remporté ce prix prestigieux est emblématique de nos fiers traditions CGS et le rôle important de la géotechnique au Canada.

Le CGS sera parrainé trois conférences importantes en 2011. Au moment de la publication, la première conférence, 5th Canadian Conference on Geotechnique and Natural Hazards aura déjà eu lieu a Kelowna. La prochaine est la Slope Stability Conference en Septembre à Vancouver. Une de mes collègues d'université, Dr. John Simmons qui a fait son doctorat à l'Université d'Alberta dans les années 1970, va faire le trek de Queensland pour présenter un document. Cela met en évidence le double objectif de nos conférences; la diffusion des dernières avancées techniques et le renouvellement de l'amitié et la collaboration technique. À la suite de ca sera la Conférence PanAm-CGS à Toronto, possiblement la conférence la plus importante parrainée par le CGS dans les deux dernières décennies. Notre comité organisateur travaille fort et s'attend à un succès de la Conférence. Même si, cette conférence sera l'occasion de faire valoir les capacités canadiennes en géotechnique. Si vous choisissez parmi les conférences internationales



à participer cette année, ce devrait être votre choix.

Venez à vos présentations des sections locales pour apprendre sur les nouvelles technologies et échanger des idées. J'espère de voir des membres CGS en grand nombre à la Conférence PanAm-CGS et j'espère que vous trouvez notre nouveau site web aussi efficace que nous espérons qu'il soit.

From the Society

Call for Nominations for Awards from the Engineering Institute of Canada (EIC)

Canadian Geotechnical Society (CGS) members are invited to submit nominations for EIC Awards to the Society Secretariat (cgs@cgs.ca) or the Secretary General (vsowacgs@dccnet.com) by not later than August 1, 2011.

Members of the Society are eligible for awards, prizes and honours from the Engineering Institute of Canada, from any of the member societies of EIC (www.eic-ici.ca), and from other institutions. By EIC Policies, all candidates nominated by CGS members to EIC awards must be members of the CGS.

The CGS Executive Committee reviews all nominations submitted by members, as well as other possible candidates by not later than August 15, 2011. Supporting documents are then prepared and forwarded to the Honours and Awards Committee of EIC for consideration. All constituent societies of EIC participate in this program. More information on the procedure, details and schedule for EIC honours and awards can be found in Section D-3 of the Canadian Geotechnical Society's Awards and Honours Manual. This information is available to CGS members in the Members Section. CGS members can log-in at http://cgs.ca/login.php then proceed to Online Member Resources, find CGS Manuals, and then proceed to the Awards and Honours Manual.

Members of CGS are eligible for the following EIC honours and awards:

- The **Sir John Kennedy Medal** is the most senior award of the Institute. This medal is awarded in recognition of outstanding merit in the engineering profession, or of noteworthy contributions to the science of engineering or to the benefit of the Institute.
- The **Julian C. Smith Medal,** established in 1939 by a group of senior members of the Institute to perpetuate the name of a Past President of the Institute. The medal is awarded for "achievement in the development of Canada".
- The **John B. Stirling Medal** was established in 1987 through the generosity of E.G.M. Cape and Company Ltd. to honour a former President of the Company who was President of the Institute in 1952. It is awarded "in recognition of leadership and distinguished service at the national level within the Institute and/or its Member Societies".

- The **Canadian Pacific Railway Engineering Medal** was established in 1988. The medal is presented “in recognition of leadership and service over many years at the regional, branch, section or equivalent levels, within the Institute or its Member Societies”.
- The **K.Y. Lo Medal** was created in 1998 and is awarded “to a member of the EIC who has made significant engineering contributions at the international level. Such contributions may include:
  - ◊ promotion of Canadian expertise overseas;
  - ◊ training of foreign engineers;
  - ◊ significant service to international engineering organizations;
  - ◊ advancement of engineering technology recognized internationally”.
- **Fellowship of EIC** (**FEIC**). A member of CGS, of at least 45 years of age, can become a Fellow of the Institute on the grounds of excellence in engineering practice and exceptional contributions to the well being of the profession and to the good of the society.
- **Honorary Membership**. The Council of the EIC may elect to Honorary Membership in the Institute, non-members who are not engineers but who have achieved distinction through service to the profession of engineering.

Provided by Victor Sowa, Secretary General

Cross Canada Lectures

The Canadian Geotechnical Society is now accepting suggestions of potential speakers for future Cross Canada Lecture Tours. Since 1965, more than 80 tours have been organized. They have included lectures by approximately 40 Canadian speakers and a similar number of overseas speakers, with a balance among consultants, academics and government engineers and geoscientists.

Information about the tours can be found in the Society’s Awards and Honours Manual (http://www.cgs.ca/cgsdocuments/). Please forward your suggested nominations of speakers to Dr. John Sobkowicz, Vice President, Technical, JSobkowicz@thurber.ca

Funding for the lecturer’s travel is supported by contributions from industry to the Canadian Foundation for Geotechnique (http://www.cfg-fcg.ca). Costs in the visited cities are paid by the local Sections of the Society.

Upcoming Conferences

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 Geotechnical Society - Corporate Sponsors

The Canadian Geotechnical Society (CGS) has an important Corporate Sponsorship program. The CGS is pleased to acknowledge the following 2010 Corporate Sponsors listed in alphabetical order:

- Geo-Slope International Ltd.
- Golder Associates
- Insitu Contractors Inc.
- Knight Piesold Consulting Engineers
- Merlex Engineering Ltd.
- Morsky Industrial Services
- Naviq Consulting Inc.
- Norwegian Geotechnical Institute
- Reinforced Earth Company
- Thurber Engineering Limited

The Canadian Geotechnical Society (CGS) represents geo-professionals across Canada as the primary technical society for access to technical information, technical journals and other related activities. The Society has a membership exceeding 1200 engineers, hydrogeologists, geologists and other related professionals. Members are employed in consulting companies, universities, resource development companies, government and non-government agencies, and others.

The CGS offers companies an opportunity to receive services and benefits through a Corporate Sponsorship program. The benefits include the following:

- Use of the CGS logo on corporate literature in combination with the following phrase "Corporate Sponsor of the Canadian Geotechnical Society".
- Recognition as a Corporate Sponsor of the Society in Geotechnical News, and at the annual geotechnical conference.
- Recognition as a Corporate Sponsor on a new page of the CGS Website, including inclusion of the company's logo and a 'hotlink' from the CGS website to the company website.

To view the CGS Corporate Sponsorship program on the CGS Website, and for additional information, refer to http://www.cgs.ca/sponsors.php?lang=en

The sponsorship program provides a significant opportunity to support the Canadian Geotechnical Society and gain additional exposure for your company. For more information on CGS Corporate Sponsorship or to sign up, please contact:

Wayne Gibson, P.Eng.
Administrator
Canadian Geotechnical Society
8828 Pigott Rd
Richmond BC V7A 2C4
Tel: 604-277-7527
Toll free: 1-800-710-9867
cgs@cgs.ca

Editor

Phil Bruch, P.Eng.
Principal, Senior Geotechnical Engineer
Golder Associates Ltd.
1721 – 8th Street East
Saskatoon, SK S7H 0T4
Tel.: 306-665-7989,
Fax: 306-665-3342,
email: Phil_Bruch@golder.com

Join us in Toronto this October

2011 Pan-Am CGS Geotechnical Conference

October 2-6 octobre 2011, Toronto, Ontario, Canada

www.panam-cgc2011.ca

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

64e conférence géotechnique canadienne et 14e conférence panaméricaine sur la mécanique des sols & l'ingénierie géotechnique

Be sure to visit us in Toronto this October when CGS will join with ISSMGE to host the 2011 Pan-Am CGS Geotechnical Conference. The Technical Committee has received over 500 technical paper submissions and is on track to deliver the largest CGS conference ever. "Geo-Innovation Addressing Global Challenges" is the theme of this integrated conference. In addition, the 5th Pan-American Conference on Teaching and Learning of Geotechnical Engineering will be held on Sunday, October 2, 2011 and will explore teaching and learning methods, as well as the implementation of industrial practice sessions into the classroom.

2011 PAN-AM CGS CONFERENCE PROGRAM HIGHLIGHTS WILL INCLUDE:

R M Hardy Address presented by Dr. K Y Lo (University of Western Ontario)

Casagrande Lecture by Dr. Kerry Rowe (Queen's University)

750+ delegates and more than 400 technical and special presentations over three days!

Local Colour Night at the Royal Ontario Museum and the 4th annual CGS Gala Awards Banquet

Technical Themes

- Retaining walls
- Ground improvement/ remediation
- Geoengineering for development & education
- Geoenvironmental engineering
- Climate change & geohazards
- Mining & rock mechanics
- Buried structures & subsurface systems
- Behaviour of unsaturated soils
- Earthquake engineering & geophysics
- Geotechnics for energy exploitation
- Laboratory testing/ In situ testing
- Shallow foundations
- Deep foundations
- Embankments and dams
- Hydrogeology and seepage
- Transportation geotechnics
- Permafrost engineering
- Mine waste disposal
- Landslides
- Probability and reliability-based design

The conference will be held at the Sheraton Centre Toronto in downtown Toronto, Ontario.

Please see the conference web site at **www.panam-cgc2011.ca** for detailed conference information and to register online.

Online delegate registration is now available – be sure to register early to take advantage of advance pricing discounts!

TECHNICAL TOUR (Thursday, October 6)

Niagara Region Geotechnical Highlights including Ontario Power Generation hydro plant, St Lawrence Seaway Museum and a winery tour/lunch

SOCIAL PROGRAM HIGHLIGHTS

Opening Icebreaker/Trade Show Reception

Local Colour Night in support of the Canadian Foundation for Geotechnique at the ROM

Gala Awards Banquet on October 4th

Platinum Sponsors:

ConeTec · Rapid Impact Piers Ltd · EBS · Geopac · coffey geotechnics · BGC · GEO-SLOPE International · Golder Associates · MEG · Reinforced Earth · Stantec · amec

GEO-INSTITUTE

Visit the Geo-Institute website at *www.geoinstitute.org*

G-I News

Geo-Risk

June 26-28, 2011
Intercontinental Buckhead
Atlanta, GA
www.georisk2011.org

Geo-Risk is the first major Geo-Institute specialty conference about risk assessment and management since 1996. You can hone your skills on how to explicitly consider risk and uncertainty to improve the value and scope of the services you provide. You also will be able to network with other engineers who may be experiencing the same challenges. Plus, you will visit one of America’s most vibrant cities.

This conference will present a unique opportunity for professional engineers, researchers, regulators and policymakers, educators and students to interact through a broad range of keynote lectures, technical sessions, panel discussions, short courses, and software demonstrations. Presentations will include eight invited keynote lectures delivered by leading international experts and more than 100 technical papers covering geohazards mitigation, risk assessment and management, and other topics. Additionally, a number of panel discussion sessions will be included to provide a forum to discuss timely topics, including levees and dams, public policy issues, geotechnical business risk and the future of risk, and reliability and probabilistic methods in geotechnical engineering.

Geo-Frontiers 2011…A Great Success

More than 1,900 participants attended Geo-Frontiers 2011 in Dallas, TX. The conference, focused on advances in geotechnical engineering, was co-organized by the Geo-Institute of ASCE, the Industrial Fabrics Association International, the Geosynthetics Materials Association and the North American Geosynthetics Society held under the auspices of the International Geosynthetics Society.

Geo-Frontiers featured six full-day short courses, four award-winning lectures, more than 450 papers presented in 89 technical sessions, exciting student activities and competitions, numerous networking opportunities, and a sold-out Exhibit Hall which was the scene of a celebratory opening night reception, a student Poster Session, and the popular Geo-Challenge Student Competition, in which 16 teams participated. Rensselaer Polytechnic Institute’s team, led by Faculty Advisor Tarek Abdoun, took home the Atterberg cup as the first-place winners. More than 300 students attended the conference, so there were plenty there to cheer the teams on!

The 4 award-winning lectures included:

- H. Bolton Seed Lecture, "Risk and Reward - Geotechnical Engineering and the Alberta Oil Sands" by Norbert Morgenstern, Ph.D., P.Eng.

- Peck Lecture, "Seismic Design of Underground Structures: Lessons from the Failure of the Daikai Station" by Antonio Bobet, Sc.D., P.E

.

- Terzaghi Lecture, "Seismic Measurements and Geotechnical Engineering" by Kenneth Stokoe, Ph.D., P.E., D.GE

- Mercer Lecture, "Use of Geosynthetics to Improve Seismic Performance of Earth Structures" by Junichi Koseki

Also hosted during Geo-Frontiers on March 16 was the Geosynthetic Research Institute›s annual conference (GRI-24). The conference featured 20 papers about "Optimizing Sustainability Using Geosynthetics."

View videos and order photos at: *http://content.geoinstitute.org/KL/Conferences/Geo-Frontiers2011.html*

Geo-Frontiers 2011

Advances in Geotechnical Engineering
Edited by Jie Han, Ph.D., P.E.; Daniel A. Alzamora, P.E Geotechnical Special Publications GSP 211 CD-ROM.

Proceedings of Geo-Frontiers 2011, held in Dallas, TX, March 13-16, 2011. Sponsored by Geo-Institute of ASCE; Industrial Fabrics Association International; North American Geosynthetics Society; and Geosynthetics Materials Association. This collection contains 496 peer-reviewed papers that describe advances in research and the practical application of geotechnologies, especially including geosynthetics. Papers from a memorial session to honor Bernard Myles for his contributions to the geosynthetics industry are also collected in this volume.

Geotechnical Professional Development Corner

Webinars

Load and Resistance Factor Design (LRFD) for Geotechnical Engineering Features: Design and Construction of Driven Pile Foundations
June 21, 2011
Noon - 1:30 PM (ET)
https://secure.asce.org/ASCEWebsite/Webinar/ListWebinarDetail.aspx?ProdId=18199

Tire Derived Aggregate in Geotechnical and Environmental Applications - Part V of VI - New!
June 28, 2011
Noon - 1:00 PM (ET)
https://secure.asce.org/ASCEWebsite/Webinar/ListWebinarDetail.aspx?ProdId=18179

Load and Resistance Factor Design (LRFD) for Geotechnical Engineering Features: Drilled Shaft Foundations
July 8, 2011
Noon - 1:30 PM (ET)
https://secure.asce.org/ASCEWebsite/Webinar/ListWebinarDetail.aspx?ProdId=18209

Load and Resistance Factor Design (LRFD) for Geotechnical Engineering Features: Micropile Foundations July 29, 2011
11:30 AM - 1:00 PM (ET)
https://secure.asce.org/ASCEWebsite/Webinar/ListWebinarDetail.aspx?ProdId=18227

For more webinars:
https://secure.asce.org/ASCEWebsite/WEBINAR/LISTWEBINAR.aspx

Seminars

Finite Elements in Geotechnical Engineering
June 9-10, 2011
Millennium Hotel Cincinnati
Cincinnati, OH
https://secure.asce.org/ASCEWebSite/Webinar/ListSeminar.aspx?CatCode=CED-GEOT

Instrumentation & Monitoring Bootcamp: Planning, Execution & Measurement Uncertainty for Structural & Geotechnical Construction Projects
Sep 22 - 23, 2011
Hyatt Place Boston Medford
Medford, MA
https://secure.asce.org/files/estore/17984/Instrumentation.pdf

For more seminars:
https://secure.asce.org/ASCEWebSite/Webinar/ListSeminar.aspx?CatCode=CED-GEOT

First U.S.-India Workshop a Success

The first U.S.-India Workshop about Global Geoenvironmental Engineering Challenges was held in New Delhi, India on November 7, 2010 in conjunction with the 6th International Congress on Environmental Geotechnics (6ICEG), held every four years under the sponsorship of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE).

A total of 17 U.S.-based participants met with their 20 Indian counterparts to discuss several key research initiatives, including sustainable waste management, green and sustainable remediation, renewable/alternative energy, and global climate change and greenhouse gases.

The participants also visited the Okhla landfill that is located in the midst of a residential neighborhood. There they observed some of the operational challenges and deficiencies not commonly seen in the West: the absence of a leachate collection system, a

Workshop Participants

methane/landfill gas collection system, or the use of daily cover. Additionally, no operational provisions are in place for waste diversion or segregation.

By combining the perspectives of the Workshop participants, researchers and professionals may collaborate and develop an understanding of the geoenvironmental challenges and needs of the developing world. Through combined efforts, researchers and professionals will have an increased ability to meet these challenges and mitigate the global effects of these challenges.

Funding for the Workshop, as well as the participants, was graciously provided by the National Science Foundation, the Geo-Institute, Indian Geotechnical Society, and India's Department of Science & Technology. For information *http://tigger.uic.edu/~kreddy/GGEC/*

Seeking International News?

A $15 ISSMGE membership will help you learn about international geo-professional news and information. The International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) promotes the advancement and dissemination of knowledge in the field of geotechnics and its engineering and environmental applications, through conferences, technical committees, and member societies. The Geo-Institute is the U.S. Member Society of ISSMGE. Former G-I President, Jean-Louis Briaud, president of ISSMGE, encourages you to become an ISSMGE member. ASCE members: Join by enrolling on your annual ASCE renewal form; logging in to your member account at *www.asce.org;* or calling 800.548.2723. An ISSMGE membership is already included in a Geo-Institute-only membership. For more information: *www.issmge.org/*

Call For Papers for Kulhawy GSP
Abstracts due: June 30, 2011

Sponsored by the Geo-Institute, ASCE will publish a geotechnical special publication (GSP) titled "*Foundation Engineering in the Face of Uncertainty*" (subtitled "*Site Heterogeneity, Property Variability, Risk, and Reliability-Based Design*"). This GSP honors **Professor Fred H. Kulhawy, Ph.D., P.E., G.E., D.GE, Dist.M.ASCE.**

Mohamad Hussein, Kok-Kwang Phoon, and **James Withiam** will serve as editors. Abstracts for proposed papers dealing with topics related to the main theme are being considered. Possible topics are:

Geologic modeling for ground characterization; Spatial variability (natural ground, modified ground); Test measurement errors (laboratory/field); Transformation uncertainties pertaining to design properties; Soil/rock property statistics (distributions, correlations); Model/bias factors in design equations; Probabilistic/reliability methods; Random finite element methods; Limit state design (ultimate, serviceability, economic, etc.); Reliability-based design (RBD); Case histories; and more.

Finished manuscripts are expected in July 2012. The GSP will be released at a specialty symposium during the G-I's 2013 annual meeting. All papers will be subjected to the standard ASCE technical papers review process. E-mail your 300-word abstract to Mohamad H. Hussein at: *MHussein@pile.com.*

International Events Endorsed by the Geo-Institute

GeoHunan International Conference II: Emerging Technologies for Design, Construction, Rehabilitation, and Inspections of Transportation Infrastructures
June 9-11, 2011
Zhangjiajie (Hunan Province), China
http://tti.tamu.edu/conferences/geohunan11/

5th International Conference on Debris Flow Hazards Mitigation
June 14-17, 2011
Padova, Italy
www.geoscienze.unipd.it/~5th-DFHM/index.htm

IS-Seoul 2011: Fifth International Symposium on Deformation Characteristics of Geomaterials
August 31-September 3, 2011
Seoul, Korea
www.isseoul2011.org/

Members in the News

Donald Appointed Terracon's National Director of Geotechnical Services

Victor Donald, P.E., was appointed national director of Geotechnical Services for **Terracon**. Donald is a senior vice president and a senior principal with the firm, working out of Terracon's Baton Rouge, LA office. He was president of Aquaterra when it joined Terracon in 2009. In addition to his current role as division manager of the Gulf Coast Division, Donald will provide leadership and direction to the geotechnical service line. He has a master's and bachelor's degree in civil engineering from Louisiana State University. Throughout his 30 years of professional experience, he gained an extensive background in geotechnical engineering and works to promote the geotechnical profession and its service to the design and construction industry.

Dr. Dennis Grubb.

Grubb Hired as Environmental Technology and Sustainable Geotechnics Director

Dennis G. Grubb, Ph.D., P.E. recently joined CETCO® in the newly created dual Directorship of Environmental Technology and Sustainable Geotechnics. In this role, Grubb will expand CETCO's capabilities to assist clients with leveraging opportunities related to the beneficial reuse of industrial byproduct and recycled materials.

"Dennis was championing the concept of beneficial reuse years before nearly anybody else had even heard of it, and he is indisputably one of the most knowledgeable and experienced experts in this emerging arena," said Archie Filshill, Ph.D., president of CETCO Contracting Services Company. "His diverse geotechnical, environmental and chemical engineering skills will add a new dimension to our services that is truly unique in the industry, allowing us to evaluate sites in a way that saves our clients money and provides a foundation for green and sustainable construction. and beneficial reuse to the benefit of our clients, both existing and new."

G-I Members Influencing Engineering

The G-I proudly recognizes the following members for distinguishing our profession by being a part of *International Water Power & Dam Construction* magazine's "Top 20 list" who made the biggest difference to the sector over the last decade.

George Annandale, Ph.D., D.WRE, F.ASCE was the first engineer to develop and publish a scientifically-defensible and validated approach to predicting scour of rock. He published a seminal paper in 1995 that forms the basis of the Erodibility Index Method and the understanding of rock scour that developed since then. Over the last 10 years, this method has matured through testing, case study validation and refinement, and has been internationally accepted by the engineering profession for use in the design of new dams and for developing mitigation designs ensuring the safety of existing dams.

John Dunnicliff, P.E., Dist.M.ASCE, a consultant in geotechnical instrumentation, has taught more than 100 CPD courses worldwide on geotechnical instrumentation. He is author of the book "Geotechnical Instrumentation for Monitoring Field Performance." He has been the editor of *Geotechnical Instrumentation News* (GIN), in the North American magazine *Geotechnical News* since 1984, and in 2010, was awarded ASCE Distinguished Membership.

Paulo Cezar Ferreira Erbisti, P.E. is the author of the textbook, "Design of Hydraulic Gate," which describes the principal aspects of the design, manufacture, installation and operation of hydraulic gates. This Brazilian engineer has contributed greatly to hydropower/dam engineering in the last 35 years in Brazil, Venezuela, and in several other countries of the world.

During his 30-year career in Canada where he worked with BC Hydro and Klohn Crippen Consultants, and in Ireland, **Desmond Hartford** has been developing the knowledge base and promoting new and modern approaches to the assessment and management of dam safety. He was one of the first in the dam engineering community to recognize that the risk-informed and systems-based approach can significantly improve the decision-making process in dam safety.

Professor Alfread 'Skip' Hendron, Ph.D. works as full-time consultant on the design and construction of dams around the world with active projects in Africa, Southeast Asia, and North and South America. Most notably, he is the primary consultant to the U.S. Federal Energy Regulatory Commission (FERC) which has regulatory authority over all non-federally owned hydroelectric dams in the US.

David E Kleiner, P.E. was the head of the geotechnical group in Harza Engineering in Chicago, IL, where he directed the design of many world class earth and rock fill dams. He also participated in other dam projects reviewing and defining the foundation for the dam (Olivenhain and Mossyrock in the U.S. for example), or as member of review boards or panel of experts (Toulnustouc in Canada, and others).

Harald Kreuzer, Ph.D., a Swiss consulting engineer, has participated in a number of international events and has served on numerous panels worldwide, particularly concerning dam safety issues. His vast experience and knowledge have made a great contribution in addressing dam safety issues in a number of countries, including Sri Lanka.

Drew Floyd, P.E., Appointed a Vice President of Moretrench

Drew Floyd.

Specialty geotechnical contractor **Moretrench** announced that **Drew Floyd, P.E.**, was recently appointed vice president. Floyd is a graduate of Michigan State University, where he earned a Master of Science in Civil Engineering. He has more than 24 years of specialty geotechnical construction experience and joined Moretrench in 2004 as Regional Manager for the New England area. In addition, he has oversight responsibility for work performed through the company's Pittsburgh, PA office. Floyd will continue in both of these capacities while assuming his corporate duties as vice president.

Kovacs Elected to ESWP Board

John W. Kovacs, P.E., PMP, D.GE, F.ASCE, was elected to the board of directors of the Engineers' Society of Western Pennsylvania (ESWP). Kovacs is a vice president and regional office manager of Gannett Fleming, an international planning, design, and construction management firm. He is based in the Pittsburgh, PA, office.

Founded in 1880, the Society is one of the country's oldest professional organizations. The Pittsburgh-based

John W. Kovacs.

group works to advance the professions of engineering, architecture, and applied sciences through technical activities, public service participation, and social organizations. The group also is committed to stimulating interest in the next generation of engineers with student outreach programs.

Kovacs holds a bachelor of science in civil engineering from Carnegie Mellon University, a master of science in civil and environmental engineering from the University of Pittsburgh, and a master of business administration from the University of Pittsburgh's Katz Graduate School of Business.

He is a licensed professional engineer in Alaska, New Mexico, Ohio, Pennsylvania, and West Virginia and a Diplomate of geotechnical engineering through the Academy of Geo-Professionals. He also is a fellow of the ASCE and holds leadership roles in the American Council of Engineering Companies of Pennsylvania and Carnegie Mellon University's Department of Civil and Environmental Engineering Alumni Advisory Board.

In Memoriam Pedro A. de Alba (1939-2011)

Professor Pedro A. de Alba passed away on Sunday, February 20, 2011.

Pedro A. de Alba.

Born in Chihuahua, Mexico on April 2, 1939, he obtained his BSCE in 1964 from the National University of Mexico. Following his undergraduate degree, he spent two years working with Dr. Leonardo Zeevaert, a well-known consulting engineer, on foundation problems in Mexico City. de Alba later obtained his MS and Ph.D. at the University of California, Berkeley under Dr. H. Bolton Seed working on liquefaction of sands during earthquakes. He later worked as a senior engineer for Shannon & Wilson in Burlingame, CA before joining the University of New Hampshire (UNH) in 1977 where he taught for 33 years. His professional interests remained in experimental techniques for measuring the dynamic response of soils. A memorial service was planned for early spring. In lieu of flowers, donations can be made towards a scholarship that will be established in his name for a student with an interest in geotechnical engineering and a passion for the arts.

Donations should be addressed to:
The Pedro A. de Alba Scholarship
University of New Hampshire
Department of Civil Engineering
Kingsbury Hall
33 Academic Way
Durham, New Hampshire 03824

Editor

Linda R. Bayer, IOM, Manager
Geo-Institute of ASCE
1801 Alexander Bell Drive
Reston, VA 20190-4400
Tel: 703-295-6162
Fax: 703-295-6351
email: lbayer@asce.org

Geotechnical Instrumentation NEWS

Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the sixty-sixth episode of GIN. Two articles this time, and three more one-pagers about web-based data management software.

New Website for GIN

The new website is www.geotechnicalnews.com/instrumentation_news.php. It has an index of GIN articles that are on the web, 83 downloadable articles, and guidelines on how to submit articles to me for future GINs.

On My Soapbox

I'm returning to a favorite topic—who should be responsible for monitoring and instrumentation during construction? By this I mean the tasks of buying and installing instruments, and collecting and interpreting data. As I've claimed many times, if significant decisions are to be based on the monitoring data, it is imperative that data quality is maximized. I contend that these four tasks should **NOT** be assigned to general construction contractors on a low-bid basis because they may not have the greatest interest in ensuring maximum quality. My following article gives four specific reasons for assigning responsibility for these tasks to personnel selected by the project owner or designer and under direct contract with the project owner.

I appreciate that, when considering most readers of GIN, I'm preaching to the converted. But we need to do all we can to get this message to owners (and project managers in design firms, who supposedly have the owner's interests at heart) that it's in their interest to adopt the recommendations in the article.

Displacement Monitoring by Terrestrial SAR Interferometry

New techniques are being developed for monitoring displacement without use of traditional geotechnical instrumentation. One of these is terrestrial synthetic aperture radar interferometry. Here's an article by a colleague from Italy.

The companies listed in Table 1 provide terrestrial SAR interferometry services. If you know of others, please tell me.

Table 1. Companies providing terrestrial SAR interferometry services

Company Name and Country	Website
Aresys S.r.l., Italy	www.aresys.it
IMG S.r.l., Italy	www.img-srl.com
NHAZCA S.r.l., Italy	www.nhazca.com

I hope to have other articles about new remote techniques in future GINs, such as:

- Satellite synthetic aperture radar interferometry
- Robotic total station able to monitor surfaces such as asphalt and concrete, using a reflectorless distancemeter.
- Airborne laser scanning by Lidar (Light Detection and Ranging)

Web-based Data Management Software

David Cook's article "Fundamentals of Instrumentation Geotechnical Database Management – Things to Consider" was in December 2010 GIN, pp 25-28. March 2011 GIN, pp 34-40, included seven one-page articles by suppliers of the software. Here are three more, by Durham Geo Slope Indicator, Roctest and Soldata.

Rick Monroe of Durham Geo Slope Indicator, whose article about their Atlas web-based data management software is on page 31, has sent me the following additional valuable recommendation, about response time:

David Cook defined response time as the delay between data collection and data presentation. Suppose we collect a reading, send it to the Internet via our cell phone, and then see a graph about five seconds later. That would be a response time of five seconds. Granted, David Cook was thinking about software, but that definition misses an important parameter: frequency of reading. Suppose we visit the site just once a week. Is five-seconds a still a relevant measure of response time? Let's change the definition to "Response time is the delay between the occurrence of an event and the monitoring system's first report of the event".

Now take a common scenario: in-place inclinometers are connected *to a data logger. The data logger takes readings every twenty minutes. Then, once an hour, a PC retrieves readings from the data logger and forwards a data file to the monitoring system. The monitoring software checks the readings and, seconds later, issues an alarm. In this scenario, response time could be as long as one hour. Is that good enough?*

If the intent of monitoring is evaluating performance, then a response time of minutes or even hours is probably acceptable. Alarms, in this case, are meant to focus attention on disturbing trends so that corrective actions can be taken. However, if intent of monitoring is to warn of a sudden event, such as a mudslide or a rockfall, then a response time of just a few seconds is required. This is the domain of dedicated, real-time monitoring systems with rapid reading rates, in-logger processing, and on-site alarms.

Fiber-Optic Sensing Systems

In December 2010 GIN, page 32, I tabulated eight commercial sources of fiber-optic sensing systems. Table 2 gives three more:

Table 2. More commercial sources of fiber-optic sensing systems

Company Name and Country	Website
Fibersensing, Portugal	www.fibersensing.com
Laser Solutions, Russia	www.lscom.ru
Marmota, Switzerland	www.marmota.com

Instrumentation Courses in Florida

There appears to be ongoing interest in these courses—for the April 2011 course there were 76 registrants from 14 different countries. The next course is planned for March or April 2013. Information will be on http://conferences.dce.ufl.edu/geotech in late summer next year.

Next International Symposium on Field Measurements in Geomechanics (FMGM)

As many of you will know, FMGM symposia are organized every four years, the previous one being in Boston in September 2007. They are "the places to be" for folks in our club. The next FMGM will be in Berlin, Germany on September 12-16, 2011. Information is on www.fmgm2011.org. I've just seen the detailed program which, at the time of writing, is not yet on the web. LOTS of papers and presentations about new and emerging technologies! Worthwhile to join us.

Corporate Changes

There have recently been three of these:

Applied Geomechanics

Founded in 1982 by Dr. Gary Holzhausen, Applied Geomechanics Inc. (AGI) began as a tiltmeter manufacturing company in Santa Cruz, California. After 25 years in the manufacturing industry, AGI was purchased by Pinnacle Technologies, a wholly owned subsidiary of Carbo Ceramics (NYSE:CRR). In the summer of 2007 AGI moved its headquarters from Santa Cruz to San Francisco, California, joining Pinnacle's base of operations. Pinnacle, a service company working in the oil and gas sector, purchased AGI to expand its instrumentation services into the civil engineering and mining markets. As a result of the merger, AGI added a variety of cutting-edge technologies, such as precision GPS and fiber optics, to its instrument and service product lines. In the fall of 2008, Carbo Ceramics sold Pinnacle Technologies to Halliburton, but retained AGI to further its growth.

Today AGI continues to sell precision equipment and services to a range of markets including volcanology, mining, heavy construction, bridges, and astronomy. AGI is currently headquartered in San Francisco, California with satellite offices in Denver, Chicago and Boston. Current management comprises: Gary Holzhausen, General Manager; Jeff Keller, Sales Manager; Jeff Crook, Engineering Manager; Tom Weinmann, Manager of Structural Health Monitoring; Alan Jones, Manager of Special Projects. For more information, please visit our website at www.geomechanics.com.

Durham Geo Slope Indicator

Durham Geo Slope Indicator is a leading manufacturer of geotechnical instruments, materials testing equipment, and environmental pumps. The company has ISO-certified manufacturing and R&D operations in Georgia and Washington states in the USA, and its products are used worldwide by consulting engineers and scientists, universities, government agencies, research laboratories, and civil and environmental construction companies.

In September 2009, DGSI became part of Nova Metrix LLC further strengthening the brand and significantly expanding the company's reach. For more information on Nova Metrix and Durham Geo Slope Indicator, please visit the company's websites at www.nova-metrix.com and www.slopeindicator.com.

Roctest

On December 10th 2010, Nova Metrix LLC, through a wholly owned subsidiary ("Nova Metrix"), completed the acquisition of Roctest Ltd. and its subsidiaries, Smartec SA, Telemac SAS, FISO Technologies Inc. and EnOmFra SAS ("Roctest").

Nova Metrix is a privately held company based in Woburn, Massachusetts, USA. Nova Metrix, through its subsidiaries and affiliates, designs, manufactures and markets test and measurement instrumentation solutions.

Nova Metrix, which also owns Durham Geo Slope Indicator, represents one of the largest producers and suppliers of instrumentation solutions for geotechnical and structural health monitoring. Nova Metrix has combined the extensive experience and expertise in traditional sensing techniques, fiber optic sensing, system integration, and data analysis.

For more information on Nova Metrix and Roctest, please visit our websites at www.nova-metrix.com and www.roctest-group.com.

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.

Na zdravie (Slovakia)

Who Should be Responsible for Monitoring and Instrumentation During Construction?

John Dunnicliff

Introduction

We all know that geotechnical construction of is not an exact science, and that therefore monitoring often plays a crucial role in ensuring that the project site and surrounding properties are safe, and meet the designer's intent. Monitoring often includes the use of geotechnical instrumentation. If significant decisions are to be based on the monitoring data, it's imperative that data quality is maximized.

If instrumentation is used, the tasks include:

1. Buying instruments
2. Installing instruments
3. Collecting data
4. Interpreting data

How can we ensure that these tasks are assigned to the people who are most likely to maximize quality?

The Golden Rule

The golden rule is: **The people who have the greatest interest in the monitoring and instrumentation data should be given direct responsibility for obtaining the data.** Or put another way, who has the motivation to do these nit-picking tasks with enough care?

Who has the motivation to do these tasks with enough care?

Who are the Candidates for Task Assignment?

They are the staff of:

- The project owner
- The project designer
- The construction manager
- The general construction contractor
- Possibly a design/build contractor
- Often a specialist geotechnical subcontractor.

The selection depends on the specifics of each project, on who has "the greatest interest".

The selection depends on who has the greatest interest

If general construction contractors, design/build contractors or specialist geotechnical subcontractors (with the agreement of the general construction contractor) have initiated the monitoring program, clearly they have the greatest interest, and all's well. But if the program has been initiated by the designer of the project, personnel in these three organizations may not have enough motivation to ensure quality. Let's look at the options for this situation.

Options for Assignment of Tasks 1, 2 and 3 when the Monitoring Program has been Initiated by the Designer of the Project.

Let's call these three tasks of buying and installing instruments and collecting data "field instrumentation services". Use of the conventional low-bid procedure, whereby these tasks are included as items in the construction bid schedule, has often led to poor quality data. Is there an alternative? Yes, there is.

There are four specific reasons for assigning responsibility for field instrumentation services to personnel selected by the project owner or designer and under direct contract with the project owner.

There are four reasons for assigning responsibility to personnel under direct contract with the project owner.

First Reason – Quality of Data

General construction contractors may not have enough motivation to ensure quality. A few years ago a UK colleague and I put together some ideas about how to maximize quality when the monitoring program has been initiated by the designer of the project. We made a strong plea for using a qualifications-based selection procedure for field instrumentation services. If you have any interest, you can download these ideas from (www.geotechnicalnews.com/instrumentation_news.php and scroll down to the only entry for 2001). Our preferred option is that the people responsible for field instrumentation services should be selected by the project owner or designer and under direct contract with the project owner. Our publication includes many comments from the technical literature in support of a qualifications-based selection procedure, which can be useful when trying to convince decision-makers to accept this method.

Second Reason - Cost

Colleagues at Mueser Rutledge Consulting Engineers in New York discuss the issue from the viewpoint of an instrumentation subcontractor to the general construction contractor (see www.geotechnicalnews.com/instrumentation_news.php and scroll to *Geotechnical Instrumentation News, Sept. 2009*). They warn:

> *The award of instrumentation work based on the 'bottom line' includes little consideration for quality, if any at all … After the contract is awarded to a construction contractor, potential instrumentation subcontractors are invited to re-bid, so that the construction contractor can compare line item breakdowns. Instrumentation bidders revisit their costs and strip contingencies. The firm ultimately awarded the work is likely to have assumed that the more stringent specification requirements will not be enforced.*

In my own experience as an instrumentation subcontractor in USA, this "stripping" can be up to 20%. Let's look at whether owners get a fair deal if this happens. As an example, if the amount assigned for field instrumentation services in the construction contractor's bid is $800,000 the project owner pays that amount, but only receives work that costs $640,000. There's a strong message for owners there.

Third Reason – Adequacy of Baseline Data

If construction work is likely to impact on neighboring structures, and monitoring with instrumentation is required to mitigate the impact, there's another important reason for favoring a contract directly with the project owner. If field instrumentation services are included in the general construction contract, monitoring can't start until the award of that contract. In that case there's rarely sufficient time to establish adequate records of pre-construction behavior (baseline data). Structures move and groundwater regimes often change from season to season, and monitoring data cannot be interpreted correctly if adequate baseline data are not obtained.

Fourth Reason – Greater Cost and also Lack of Conformance on Multi-general Contract Projects

For multi-general contract projects, there would be one monitoring subcontractor for each construction contract, hence greater cost when compared with a single assignment.

For multi-general contract projects, the various monitoring subcontractors would probably make different selections of web-based data management software, so that contract-to-contract comparisons would be difficult. This also places a heavier burden on a construction manager needing to become simultaneously proficient in more than one system.

Recommendations for Assignment of Tasks 1, 2 and 3 (Buying and Installing Instruments and Collecting Data) when the Monitoring Program has been Initiated by the Designer of the Project.

My recommendations are given in Table 1.

Table 1. Recommendations for assignment of tasks 1, 2 and 3 when the monitoring program has been initiated by the designer of the project

Type of Monitored Data	Recommendations for Assignment of Tasks 1, 2 and 3
Pre-construction baseline data	Specialist firm under contract with the project owner
Data during construction, outside general construction contractor's work area	Specialist firm under contract with the project owner
Data during construction, within general construction contractor's work area	Either: • Construction manager, with assistance from general construction contractor for access as necessary, or • Specialist firm as assigned subcontractor, instrument suppliers as assigned suppliers (see box on next page), or • General construction contractor, with partnering and rigorous and **enforced** specifications.

Options for Assignment of Task 4 (Interpreting Data) when the Monitoring Program has been initiated by the Designer of the Project.

Clearly the people who initiated the monitoring program should have a role in interpreting the data. However, the general construction contractor MUST pursue a parallel effort, and construction documents must specify that the general construction contractor has the **primary** responsibility for interpretations and must stay on top of the data flow at all times.

Closing Comments

I know very well that it isn't easy to convince owners (and project managers in design firms, who supposedly have the owner's interests at heart) that it's in their interest to adopt the above recommendations, **but it is!** Join the campaign to ensure that the people who have the greatest interest in the monitoring and instrumentation data should be given direct responsibility for obtaining the data.

Join the campaign!

John Dunnicliff

Assigned Subcontractor and Assigned Supplier

When the *assigned subcontract* method is used for installing instruments and collecting data, the project owner or designer negotiates with specialist firms, selects one firm using a qualifications-based selection procedure, and assigns the contract to the construction contractor for administration. Payment is made on the basis of actual work done, and the cost is included in the total bid price. A line item in the bid schedule is designated as an *allowance item* and *Provide Services of Specialist Field Instrumentation Personnel* is entered in the description column. The cost estimate is included in the bid schedule. An explanation of this procedure is included in the contract documents. After contract award, the construction contractor is instructed to enter into a subcontract with the assigned subcontractor, and payment is made to the subcontractor via the construction contractor under the allowance item. The construction contractor's monthly payment requests to the owner are supported by including copies of subcontractor invoices. The cost estimate should not be regarded as a not-to-exceed figure, and the contract price should be increased by change order if needed.

Opposition to this procedure sometimes includes the concern that the subcontractor, who has been selected by the project owner or designer, is under contract with the construction contractor, hence is there uncertainty about contractual commitment? In my experience, with appropriate people-communication, this has never been a problem in the field.

When the *assigned supplier* method is used for buying instruments a similar procedure is used with another allowance item, *Furnish Instruments*. The specification states that, after contract award, the owner's representative will determine instrument descriptions, sources, quantities, and prices and will provide this information to the construction contractor. The contractor is then required to place orders, within a specified time period, and the instrument suppliers become assigned suppliers.

Displacement Monitoring by Terrestrial SAR Interferometry for Geotechnical Purposes

Paolo Mazzanti

Acronyms Used in this Article

- GPS: Global Positioning System
- RTS: Robotic total stations
- SAR: Synthetic Aperture Radar.
- SInSAR: Satellite SAR Interferometry
- TInSAR: Terrestrial SAR Interferometry
- TLS: Terrestrial Laser Scanner
- DTM: Digital Terrain Model

Introduction

The geotechnical community is looking with increasing interest at emerging technologies. Innovative techniques able to solve problems that have been unsolved for decades are now available. However, geotechnical engineers and engineering geologists must be confident on their effectiveness before applying them, and especially on the reliability of collected data. Remote sensing techniques are one of the main innovations in the field of geotechnical monitoring, since they are changing the philosophy from "contact" to "non-contact" monitoring. In other words, by remote sensing techniques, some geotechnical parameters are collected by equipment located away from the investigated area. However, ground-based remote sensing instruments such as manual or robotic total stations and GPS (Global Positioning Systems) cannot be defined as fully "non-contact" instruments since they need targets or sensors installed on the monitored ground or structure. Among the ground-based techniques, only Terrestrial Laser Scanner (TLS) and the Terrestrial SAR Interferometry (TInSAR) can be considered completely "non-contact" remote sensing techniques.

The first prototypes of TInSAR were developed at the end of the 1990s, and the first commercial equipment dates back to 4-5 years ago. Seven years experience with TInSAR has allowed me to follow this technique from its first steps to the first long-term and successful applications for complex geotechnical problems.

In what follows, the basic principles of this technique, together with a detailed description of its performance, main advantages and limitations and lessons learned from real cases will be discussed.

Theoretical Basis and Performance

The Terrestrial SAR Interferometry (Bozzano et al. 2010; Luzi 2010) is a displacement monitoring technique based on the same operational principles of Satellite SAR Interferometry (Massonet & Fiegl 1998). The SAR principle consists of a combination of several radar images collected while the emitting and receiving antennas move along a predefined trajectory (an orbit for a satellite, a route for an airplane or a rail in the case of terrestrial equipment) (Figure1). The

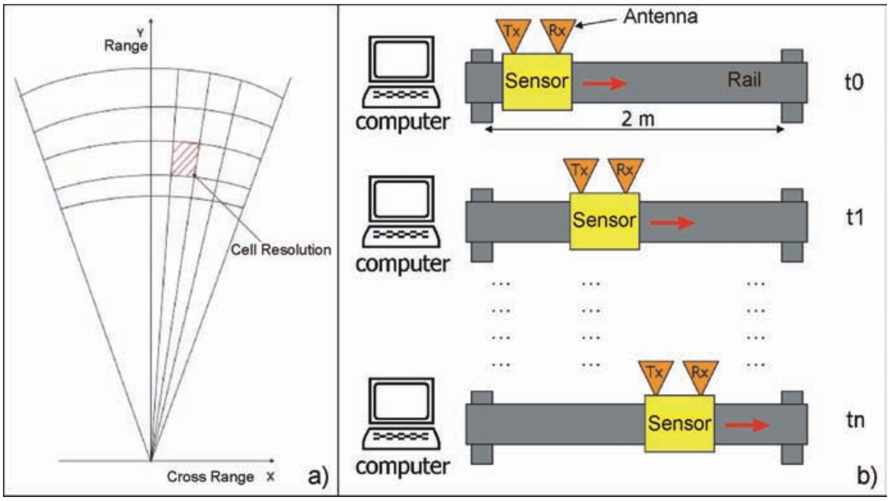


Figure 1. a) Resolution cell of RADAR maps; b) Synthetic aperture obtained by an antenna moving along a rail.

combination by the focusing technique of radar images that are acquired during the movement of the antennas allows 2D SAR images to be obtained. These images are characterized by range (instrument-scenario joining direction) and cross-range (direction normal to the range direction in the horizontal plane) resolution (Figure 1). The final SAR image consists of several pixels whose size strongly depends on the equipment features and on the radar-scenario distance.

By comparing the phase difference, i.e interferometric technique, of each pixel of two or more SAR images collected at different times, the displacement along the instrument line of sight can be estimated by using the following equation:

$$d = \frac{\lambda}{4\pi}\Delta\phi$$

where *d* is the displacement, λ is the wavelength of the radar signal and $\Delta\phi$ is the phase difference between the two acquisitions. However, additional processing aiming at remove the atmospheric noise is required. The final output of TInSAR monitoring is 2D color images where the magnitude of displacements along the instrumental line of sight, in the computed elapse of time, can be quickly identified (Figure 2). In addition, displacement time histories of each pixel of the image can be achieved.

The pixel resolution of a SAR image ranges from few decimeters to several meters (depending on the equipment and on the monitoring distance) and the displacement accuracy ranges from few tenths of millimeters to a few millimeters, depending on the operational distance and the atmospheric conditions. For example, at a distance of 1 km, commercial equipment has a range resolution of about 0.5 m and a cross-range resolution of 4 m; as regards the accuracy values ranging from 0.5 to 3 mm are reasonable at a distance of 1 km. This equipment has a maximum range capability of few kilometers and a maximum temporal frequency of images collection of few minutes. However, future TInSAR equipment is expected to be faster in data collection and smaller in size.

Advantages and Limitations

As already stated, TInSAR is one of the two “real” remote monitoring sensing techniques, since it does not require the installation of sensors or targets in the monitored area. This is probably one of the main advantages of TInSAR as the access to the monitored areas is often dangerous (e.g. active landslides), difficult (e.g. cliffs) or prohibited by local authorities, such as heritage situations. Sometimes, we are faced with movements so rapid, e.g. rapid landslides, that sensors are quickly destroyed or made unusable. In these cases remote TInSAR monitoring can be an efficient solution. An additional advantage is related to the control of an area (i.e. pixel) instead of single points identified by sensors, reflectors etc. This feature can reduce the misinterpretation, which is a frequent problem in the case of points-based monitoring. On the other hand, the analysis of an area instead of a point can also be a limitation if this area behaves in a heterogeneous way, or if the monitoring of a specific point is required. In these cases passive corner reflectors for TInSAR can be installed, thus allowing the increase of the signal to noise ratio of the pixel and also the precise identification of the monitored point.

A further advantage of TInSAR is the full operability under all lighting (day and night) and weather conditions (rainfalls, clouds, fog etc).

A significant advantage is the ability for “spatial” monitoring. This means that TInSAR can be used to simultaneously monitor the displacement of

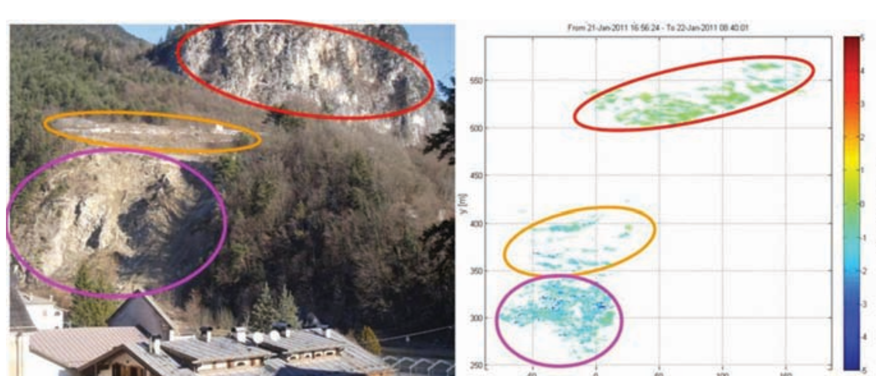


Figure 2. Picture of a slope (on the left) and TInSAR displacement image (on the right). Color ellipses enclose corresponding parts of the investigated slope.

several adjacent pixels over large areas. In other words, TInSAR images can be seen as a very dense network of adjacent sensors (i.e. pixels) collecting data simultaneously over a large area. The main practical advantages of this feature are:

- increasing the statistical reliability of monitored displacements because data are collected in several adjacent pixels
- monitoring of large areas, thus avoiding the risk of underestimating the size of the displacement area
- identification of spatial distribution and gradient of displacement.

Additional features such as the high data sampling rate (few minutes), long range efficacy (up to some kilometers) and the high accuracy in the displacement measurement make this technique a valuable monitoring solution for appropriate geotechnical problems.

However, in spite of its advantages, this technique is characterized by some limitations which must be taken into account. The difficulties in the management, processing and interpretation of data are probably the main limitations. Mistakes can be made if the technique is not used in the appropriate way and if data are not analyzed carefully. Some additional limitations related to technical features are:

- the large size of commercial equipment, having a rail of at least a couple of meters long
- the cone of view is limited to a few tenths of degrees (depending on antennas) in the horizontal and vertical planes
- the displacement can be measured only along the line-of-sight direction, i.e., the displacement monitored by TInSAR is only a component of the real displacement
- phase ambiguity, i.e. the displacement between two subsequent images can be measured without ambiguity only if the phase difference is lower than $\pi/2$ (about 4.5 mm for the typical signal frequency used by the Terrestrial SAR Interferometers).

However, the above mentioned limitations can be reduced by a careful monitoring planning (in terms of the installation site and the monitoring plan). For example, in order to optimize the displacement detection capabilities the equipment can be installed as parallel as possible to the real displacement direction. The phase ambiguity can be solved (up to a threshold velocity on the order of meters/day) by a high data sampling rate.

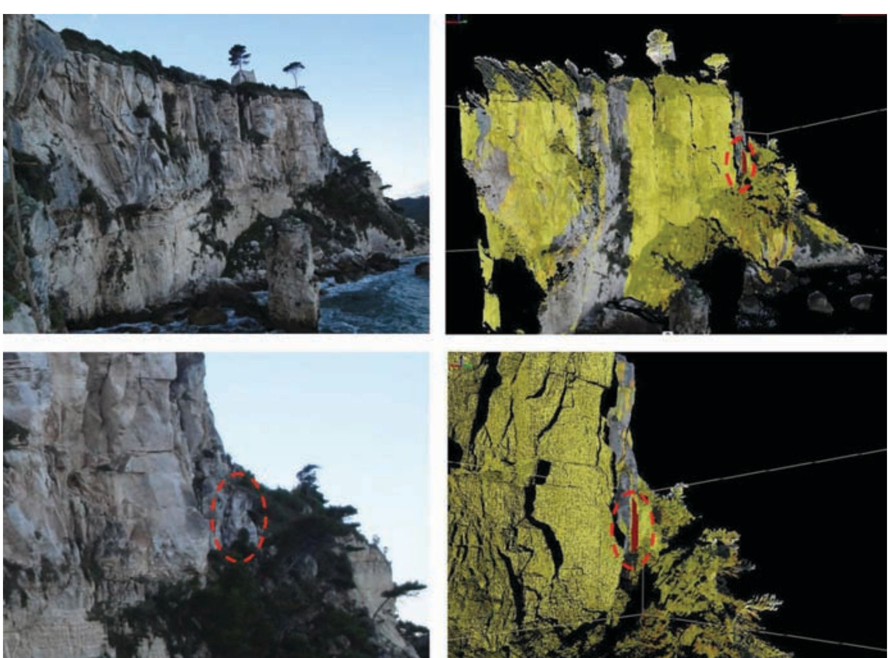


Figure 3. Picture of a costal rock cliff in the southern part of Italy (on the left). On the right, 3D displacement images achieved by the combination of TInSAR image and TLS DTM (Digital Terrain Model); yellow-green color identifies stability while red color identifies sectors affected by displacements.

Comparison with Conventional Techniques

The first comparison of TInSAR should be with Satellite SAR Interferometry (SInSAR), since they are based on the same operational principle. However, due to the different platforms (ground-based and satellite-based respectively) there are several differences between them, especially in terms of achievable results. SInSAR is a suitable technique for monitoring large areas characterized by slow movement (e.g. subsidence, volcanic structures, unstable regions etc.), while TInSAR is more suitable for the detailed and continuous monitoring of small areas, up to few square kms, that are characterized by both slow and rapid movement (e.g. single unstable slopes and cliffs, volcanic flanks etc). Also, due to the low data sampling rate (about one image per month), SInSAR is not suitable for control and continuous emergency monitoring, but is more appropriate as an investigation tool (especially if the historical database of satellite images available from 1992 is considered). In contrast, TInSAR images can be collected only after the installation of equipment.

The comparison of TInSAR with robotic total stations (RTS) is probably more appropriate because these techniques are often used for similar applications, even though they are based on different operating principles. In what follows a brief comparison between these two techniques is given. First of all, RTS is based on Laser technology, while TInSAR is based on Radar technology; i.e. RTS uses Light or Infra-Red waves while TInSAR uses Microwaves. From the practical point of view the main difference is related to the monitoring effectiveness of TInSAR with the presence of fog and clouds (not acceptable for RTS). Furthermore, RTS requires the installation of targets in the monitored area while

TInSAR is a completely remote monitoring technique. This is an important feature when faced with heritage situations and unsafe areas such as landslides.

On the other hand TInSAR can only measure line-of-sight displacements while RTS can be used for measuring the 3D displacement field.

The accuracy of displacement monitoring of the two techniques is difficult to compare, since it strongly depends on processing solutions and specific site conditions. However, experiments under ideal conditions have demonstrated that similar accuracies can be achieved.

Successful Applications and Lessons Learned

In recent years successful applications of TInSAR have demonstrated that such a technique is a very powerful and versatile solution for the monitoring of different types of geotechnical and structural engineering problems, and especially for the continuous monitoring in emergency conditions, e.g. landslides and volcano flanks. The recent application at an unstable slope overlooking an artificial lake in a mountainous region (up to 3000 m above sea level), frequently affected by fog, demonstrated the effectiveness of TInSAR under any weather conditions: a basic requirement for 24/7 emergency monitoring.

But in the author's experience, the most complex application of TInSAR has been the monitoring of a slope affected by construction of a tunnel. Because of the presence of a large and deep active landslide in rock material, continuous monitoring of the slope stability was required. Displacement monitoring by conventional on-site techniques (e.g. inclinometers, total stations, GPS) was difficult due to the geomorphology of the area and the ongoing construction work at the tunnel entrance (gabions, anchored bulkheads etc). Furthermore, a technique with minimum intervention of personnel on the slope for installing instruments or targets was necessary for 24/7 emergency monitoring. The continuous monitoring of this slope by TInSAR, from a distance of about 1 km, allowed monitoring of every type of displacement that affected the slope: excavated debris, gabions, bulkheads etc. This allowed engineering decisions to be made efficiently, such as stopping tunnel excavation as a consequence of sudden increases of slope displacement such as an increase of one order of magnitude of the velocity in a few hours. 3½ years of continuous monitoring by TInSAR, continuing to this date, demonstrated the long-term reliability of this technique and its effectiveness in monitoring both rapid and slow movements. This feature, together with the capability of monitoring without any targets on the slope, makes TInSAR particularly suitable for the monitoring of ground movements that are characterized by a high and non-homogeneous velocity field and little vegetation cover. In addition to the project just described, several cases of ground movement have been monitored by TInSAR in recent years, both for emergency and investigation purposes.

Further suitable applications of TInSAR for geotechnical problems are the monitoring of dams and mines.

But the new frontier of TInSAR is probably monitoring for investigation purposes. For example, displacement monitoring of several points over large areas by TInSAR has recently been proven for susceptibility analyses of cliffs. In this application TInSAR has been used for determining and mapping the most susceptible sectors of cliffs, slopes and man-made structures.

The monitoring of buildings and heritage situations in urban areas is a new challenge for TInSAR. On one hand there is the great advantage of having highly accurate displacement images by a non-contacting technique, but on the other hand there must be separate monitoring for vertical movements. At present, combining with conventional techniques is considered a basic requirement in such applications.

Conclusion and Outlook

Terrestrial SAR Interferometry is an emerging technique for geotechnical monitoring. Although not yet extensively used in common practice, TInSAR has been successfully proven for monitoring some geotechnical problems such as landslides and dams, and is a promising method for some others, such as cliffs and buildings. The high price of equipment and the complexity of data processing and interpretation of results can be considered the main limitations for extensive use of this technique. However, TInSAR can be more efficient than conventional monitoring, and in some cases also less expensive if rational monitoring plans are made. Private companies specializing on TInSAR already exist. Furthermore, the combination of TInSAR with other techniques such as Terrestrial Laser Scanner and robotic total stations may further strengthen its effectiveness and simplify the interpretation of results—see an example in Figure 3.

References

Luzi, G. 2010. Ground based SAR interferometry: a novel tool for Geoscience. Geoscience and Remote Sensing New Achievements, 508pp.

Bozzano, F., Mazzanti, P., Prestininzi, A., Scarascia Mugnozza, G. 2010. Research and development of advanced technologies for landslide hazard analysis in Italy. Landslides.

Massonet, D., Fiegl, K. L. 1998. Radar Interferometry and its application to changes in the earth's surface. Reviews of Geophysics, 36(4), 441-500.

Paolo Mazzanti, Chief Executive Officer, NHAZCA S.r.l. - spin-off "Sapienza" Università di Roma, Via Cori snc, 00177, Rome, Italy, Phone: +393469776508, email: paolo.mazzanti@nhazca.com.

Geoscope Web-based Data Management Software

Martin Beth, Soldata

Introduction

This is not a product presentation but rather a general paper about web-based data management software. In this short one-pager I shall first list the typical level of expectation nowadays, based on my understanding of technical specifications from all size projects in US, Europe and Asia, and then indicate some important issues and lessons learned from our experience.

The Standard Expectations

All clients nowadays expect the following:

- Data to be displayed on the Internet as soon as collected.
- Full Internet access, password protected, available on PC, tablets or mobile phones.
- Graphical site views, helping the users to understand the large flow of data coming towards them. These views should combine flexibility and simplicity, different graphs types, etc…
- Ability to integrate all types of automatic or manual data, for any type of sensor.
- Ability to carry out various calculations of the data.
- Alarms by mail and SMS.

First Important Issue: Scalability

The software should be simple and easy to use and be applicable for small sites. But it should also have the capacity to handle mega-sites such as for example Barcelona Linea 9 Metro Line, where we currently have 1.5 billion data from over 50,000 measurements points, inside a 130 Gb database.

The risk of too much data should be overcome. The system should help the user to remain in control of the data flow; it should include tools to simplify, to filter and to sort the data. This is neither easy nor obvious.

Second Important Issue: Security

What happens if something goes wrong with the monitoring system itself? The following two main features should be available:

- There should be a watchdog computer somewhere, separate from the site and from the database, checking that the monitoring system and the module in charge of sending the alarms are working properly, have operational internet access, etc….
- It must be remembered that SMS are not considered as a certified and secured system. Have you never received an SMS a few hours or even a few days after it was written? The software should include an automatic repeat mode or even an automatic escalation process until the alarm has been acknowledged.

Security against data loss is crucial. Storage of intermediate data at different steps along the data flow line should also be implemented. Furthermore the system should have the capability to process past data when restarted.

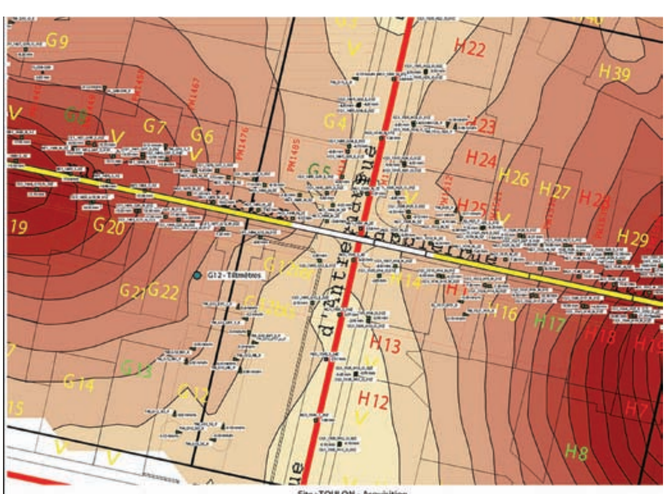


Figure 1. An isolign plot of road surface and buildings settlement, and the tunnel advance (yellow line), both updated automatically in "real time".

Third Important Issue: Data Presentation and Data Analysis

From our experience:

- 3D interactive "computer game" type site views are very nice and sexy and will add a strong positive feeling about the software. But in reality let's face it; they are not a lot of use to the engineer. Over the past 10 years and say 500 monitoring sites, we have probably used this functionality a dozen times.
- On the contrary it is of the utmost importance for the software to be able to integrate external information of parameters affecting the data, like tunnel face position, comments by the users about the data, geological log reports, grouting data, and other external event likely to influence the results. The system should include a log book, for users to enter any type of information, and it should be possible to view this information on the graphs.
- Isolign plots are also useful in grasping rapidly a global idea of the site behaviour. See Figure 1.

Martin Beth, Technical Manager, Soldata Group, c/o NCC, 12 McClane Street, Cuddy. PA, 15031, USA, (412) 860-2973, martin.beth@soldata.fr.

SHMLive Web-based Data Management Software

Daniele Inaudi, Roctest / SMARTEC

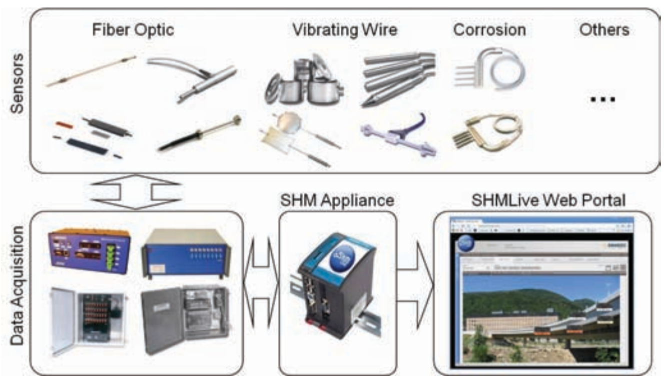


Figure 1. SHMLive System Architecture.

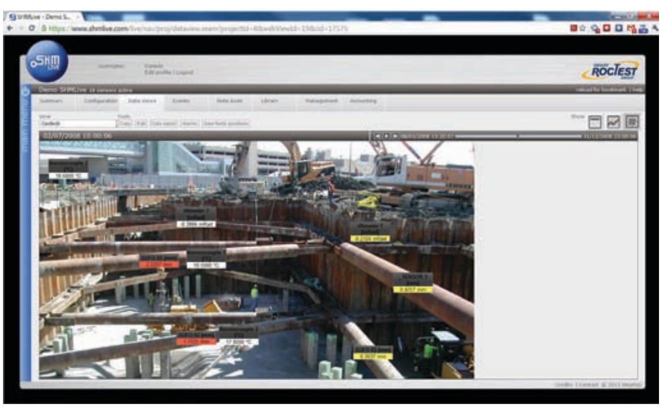


Figure 2. Map data representation example.

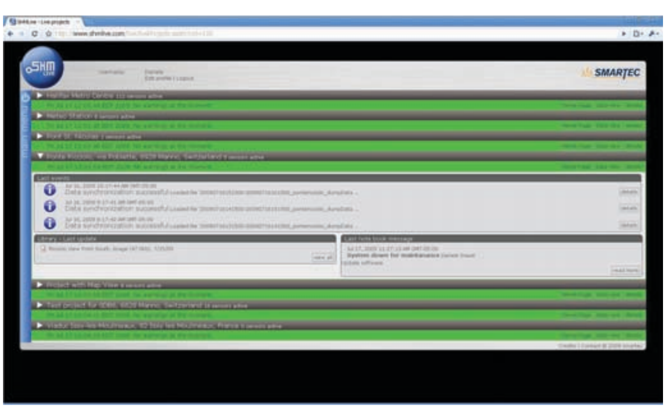


Figure 3. SHMLive project summary page.

Web-based services are becoming the new standard in reliable and cost effective mission-critical business applications, such as email, customer relationship management tools or document exchange. In the same way that it makes sense to operate your own power generation station in house, the management and publication of monitoring data is more efficiently managed by instrumentation and IT professionals, rather than civil or geotechnical engineers or owners.

The SHM-Live web portal is a secure hosted website coupled with an online database that manages and displays monitoring data in real-time anywhere in the world. SHMLive web portal is a part of Roctest's complete SHM-Live offering, which can also include full monitoring services, such as design, installation and all hardware, provided for a fixed monthly fee.

The SHM-Live database can receive data from a large variety of measurement systems and sensors such as vibrating wire instruments, fiber optic sensors, laser sensors, concrete corrosion sensors, and any type of electrical sensors. As depicted in Figure 1, data is automatically pushed to the SHMLive database from our SHM Appliance, which collects the data directly from all installed data acquisition systems, without the use of text files or other intermediate data formats. All data is stored in our secure and redundant database system, located in a data center with the highest standards of reliability and security.

Authorized users gain access to their data through an easy to use online web portal where data is available 24/7 for display and downloading to Excel and other formats. The web interface allows different levels of authorization for data access and users can easily log on with any web browser or smart phone. The SHMLive web portal allows real-time alerting and advanced data representation, enabling an unlimited number of data views in table, graph or map plots (Figure 2) with associated options such as thresholds plots, X-Y plots and color coding. It is also possible to define warnings and alerts, based on individual sensors or free mathematical formulas, combining the values of multiple sensors. Alert levels, language of the user interface and delivery methods, such as email or text messaging, can be tailored to individual user preferences.

The Web portal also serves as an information hub, allowing the storage of complementary documents, reports, alert histories and log book entries, facilitating communication among all stakeholders within the monitoring project. A summary page (Figure 3) allows a quick overview of the status of all projects to which the user has access. The web-interface can be re-branded with the user logo, with links to any external websites containing complementary data, such as webcams or meteorological data.

The SHMLive portal is accessible at www.shmlive.com.

Daniele Inaudi, CTO
SMARTEC / Roctest Group
Via Pobiette 11, 6928 Manno,
Switzerland,
daniele.inaudi@smartec.ch,
www.roctest-group.com

Atlas Web-Based Data Management Software for Instrumentation

Rick Monroe, Durham Geo Slope Indicator

Atlas - the Project Web Site

Think of Atlas as a web site that is dedicated to a project. The pages of the web site include plan views and photographs of the project and contain links to data, graphs, and reports. Users log into Atlas with their web browsers.

Atlas provides three levels of access. "Administrators" can create new projects, authorize users, and set up sensors, graphs, plan views, alarms, and reports. "Users" can see graphs and plan views, enter manually-collected readings, and add notes and photos to the logbook. "Guests" can see only selected plan views and plots.

Data Collection

Atlas provides web forms to receive manually-collected readings, a logbook to receive notes and photos, and an input folder to receive data files forwarded from data loggers.

Atlas processes incoming data to check for alarm conditions, but it stores only the original, unprocessed readings in its database. Thus readings in the database remain directly traceable to readings collected at the site.

Data Processing

When Atlas generates a graph or serves data, it always processes the original readings on the fly. This makes calculations easy to verify, and it ensures that changes or corrections to calculations take effect immediately, with no need to purge and rebuild the database with corrected readings.

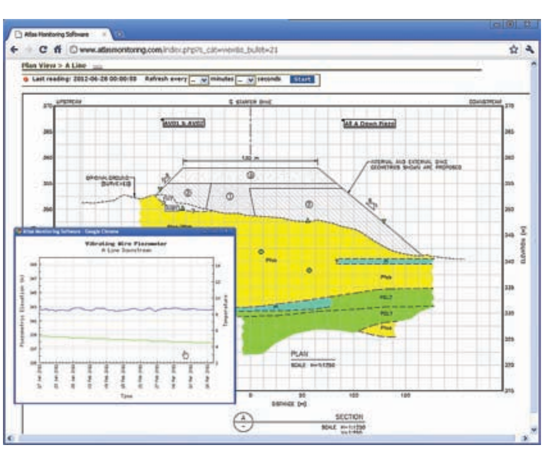
The core of the Atlas processing engine is the sensor table. It lists every sensor along with its calibration factors, unit conversions and labels, alarm limits, and processing instructions. Processing instructions accept most math functions and can reference earlier readings and other sensors. This makes it possible to calculate changes, correct for temperature and barometric pressure, and perform cumulative calculations for in-place inclinometers and beam sensors.

Data Presentation

Plan views are site drawings or photographs that show the location, current reading, and alarm status of all the sensors at a site. Sensors are represented as icons that change color to indicate their alarm status: green for normal, yellow or red for alarms. Mousing over an icon displays the current reading, and clicking on a reading calls up a trend plot. A quick look at the trend plot can reveal whether the alarm condition is the result of a trend or a transient event.

Plots present data graphically and automatically include the most recent readings. Atlas provides trend plots, profile plots, and correlation plots. Multiple Y scales allow different types of sensors to be shown on the same plot. Clicking the plot displays a table of the values used in the plot.

Reports present a daily, weekly, or monthly compilation of selected plots, data, log book entries, and photographs. Reports can be distributed automatically by email as PDF attachments.



Alarms and Notifications

When Atlas detects an alarm condition, it records the alarm in a logbook, displays an on-screen warning, and generates an alarm notification. An alarm notification is an email or sms message that identifies a sensor, the time and value of the reading, and the level of the alarm.

Atlas provides filters that help validate alarms, consolidate notifications, and delay or escalate notifications. This filtering improves user confidence in the alarm system and also prevents alarm notifications from flooding email boxes and cell phones.

Data Downloads and Archiving

Readings can be downloaded for analysis in other programs. After the user specifies sensors, a date range, and a data format, Atlas generates a text file that can be saved on a local PC and opened in a spreadsheet.

Data can be archived two ways. Archiving processed readings makes data available for historical investigations after completion of the project. Archiving the original readings provides a way to control the size of the database, though this function is rarely needed.

Software Response Time

The overall response time of a monitoring system is likely to be controlled by the rate of data collection rather than by the responsiveness of the software. That said, Atlas can serve graphs within one or two seconds, refresh plan views every few seconds, and send out alarm notifications seconds after the arrival of new readings.

Rick Monroe, DGSI,
12123 Harbour Reach Drive,
Mukilteo, WA 98275 USA,
Tel: 425-493-6200,
email:Rmonroe@slope.com

Computing in Geotechnical Engineering

Alireza Afkhami
Saeed Otufat-Shamsi

Among the geotechnical in-situ tests, Cone Penetration Test provides a large dataset which needs to be interpreted carefully by geotechnical engineers. During last decades, many researchers have been carried out on Cone Penetration Test in order to provide empirical relations between CPT data and the engineering properties of soil layers.

During this test, the soil ahead and behind the cone will influence tip resistance (qc); hence affecting the soil behavior type (particularly during the transition between two significantly different soil types such as clay and sand). In this article, Dr Robertson suggests an SBT index (Ic)-based approach to identify and remove transition zones in CPT dataset.

We would like to hear more about this concept and may be different point of views. Please send your technical articles to *Saeed@NovoTechSoftware.com*

Submission Guidelines

Manuscripts should be sent to the editor at Saeed@NovoTechSoftware.com. Normal length is 2-4 pages in the magazine page format including figures and photos. The text must be in Word file. And please define all acronyms in your technical article. Geotechnical News is principally looking for articles that are of interest to GeoFolk, written clearly and concisely and not too much trade promotion. Please note that the articles will not be "peer reviewed". For more information and the details about the format please review the guidelines for articles at: *http://www.geotechnicalnews.com/pdf/guidelines_submitbitech08.pdf*

Computing in Geotechncial Engineering of *Geotechnical News*.

Saeed Otufat-Shamsi, NovoTech Software, 4188 Hoskins Rd, North Vancouver, BC, V7K-2P5 Canada, email: Saeed@NovoTechSoftware.com

Automatic Software Detection of CPT Transition Zones

P.K. Robertson

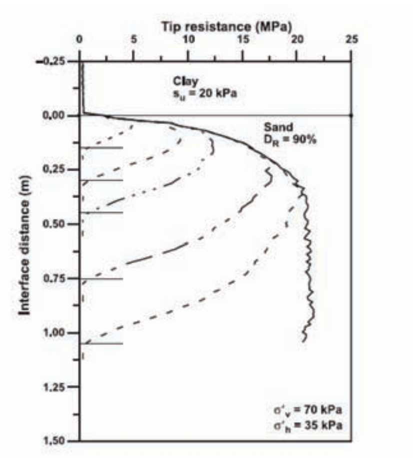


Figure 1. Numerical simulation of CPT tip resistance for a 10 cm^2 cone in dense sand layers within soft clay (after Ahmadi and Robertson, 2005).

One of the main advantages of the Cone Penetration Test (CPT) is the continuous nature of the test results that provide excellent profiles of soil type, detailed stratigraphy, and in-situ mechanical properties of the ground. However, for some applications, such as inter-layered soils, the continuous nature of CPT results can present challenges when using CPT-based software for geotechnical design.

It has long been recognized that although the CPT measures the correct cone tip resistance (qc) in uniformly weak or strong materials, the transition from one layer to another will not necessarily be registered as a sharp change in qc at the inter-layer boundary. Experimental studies (Treadwell, 1976) have shown that the cone tip resistance is influenced by the material properties both ahead and behind the penetrating cone. Hence, the cone will start to sense a change in material type before it reaches the new material, and will continue to sense the previous material after if it has entered a new material. Therefore, the CPT may not always measure the correct tip resistance in the transition zone from one soil layer to another, if the soils have strongly different cone values.

Lunne et al. (1997) note that the distance over which the cone senses an interface increases with material stiffness. Ahmadi and Robertson (2005) illustrated this using numerical analyses and confirmed that in strong/stiff soils the zone of influence is large (up to 15 cone diameters) whereas in soft soils the zone of influence is rather small (as small as 1 cone diameter). Figure 1 (after Ahmadi and Robertson, 2005) illustrates the variation of cone tip resistance from a compilation of several analyses in which the thickness of a dense sand layer in a soft clay deposit is changed. Figure 1 shows that as the thickness of the sand layer increases, the tip resistance reaches closer to the true tip resistance of the sand. For the set of soil parameters selected in the analyses (i.e. dense sand in soft clay), the results shows that the cone can reach its fully mobilized tip resistance in the dense sand if the thickness of the sand layer is equal to or greater than 1.0 m, i.e. about 28 cone diameters. Figure 1 also shows that for the initial part of the penetration profile in the sand layer, all tip resistance profiles for different layer thickness have a common transition profile. This indicates that the cone at the beginning of penetration into the sand layer responds similarly irrespective of the layer thickness. However, as the cone penetrates further, it starts to sense the upcoming new soft clay interface. The thinner the sand layer, the sooner the cone responds to the soft clay below and the smaller the maximum measured cone resistance within the sand.

The zone of influence ahead and behind a cone during penetration will influence the cone resistance at any interface (boundary) between two soil types of significantly different strength and stiffness. Hence, it can be important to identify the transition zone between different soils types to avoid possible misinterpretation. This issue has become increasingly important with CPT-based software that provides interpretation of every data point from the CPT. When CPT data are collected at close intervals (typically every 20 to 50mm) several data points are '*in transition*' when the cone passes an interface between two different soil types (e.g. from sand to clay and visa-versa).

For some geotechnical design problems, such as pile design, the existence of transition zones in a CPT profile is captured in the design methods by accounting for scale effects between the

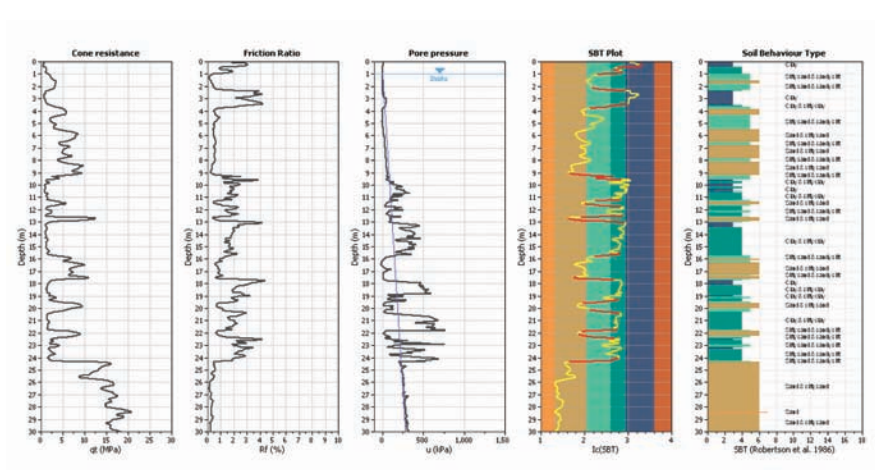


Figure 2. Example CPTu profile in inter-layered soils to illustrate transition zones (indentified in red) on the SBT I_c profile (4th column).

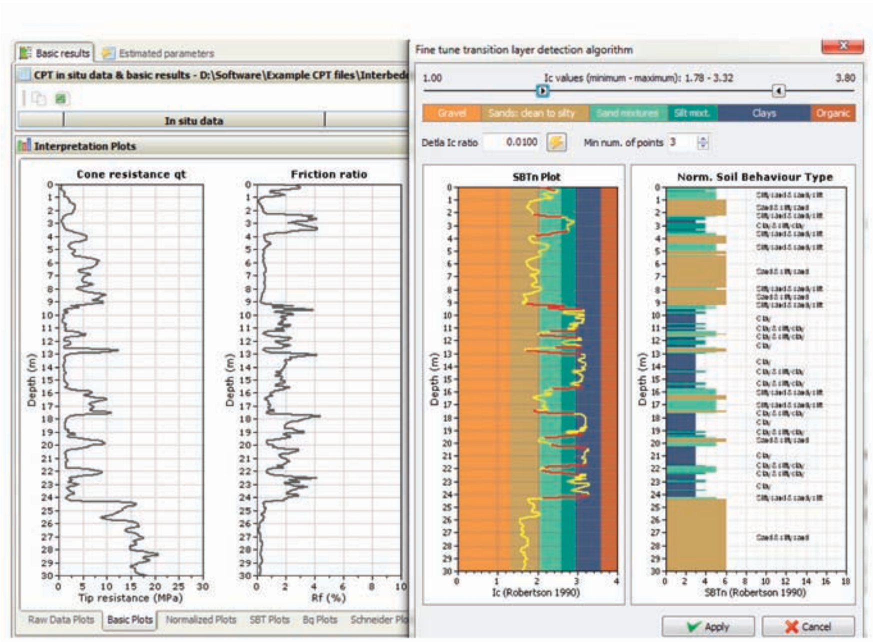


Figure 3. Transition layer detection dialog box used in CPeT-IT for the example CPTu.

cone and pile size. However, when estimating geotechnical design parameters in inter-layered deposits, the transition zones at the layer interfaces may produce misleading variations in estimated parameters. For example, either the sand close to the boundary with a soft clay layer may appear loose, or the clay next to a sand layer may appear stiff in the transition zone. In a liquefaction analysis the CPT data within the thin transition zone (e.g. from a sand to a clay) can result in a misinterpretation of soil type that may predict that the soil in the transition zone has the potential to liquefy resulting in conservative additional calculated post-earthquake deformations. Hence, it can be helpful if the CPT-based software has the ability to both indentify and remove any transition zones to evaluate their influence on any subsequent analysis. The following describes how software can be used to automatically detect and remove transition zones in CPT data.

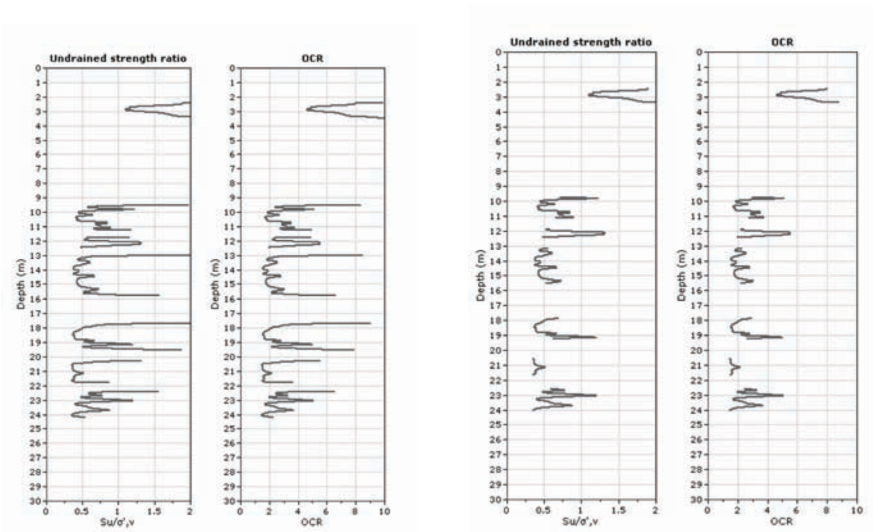


Figure 4. Example CPTu profile showing estimated undrained shear strength ratio (s_u/σ'_{vo}) and OCR (a) with transition zones, and (b) with transitions zones removed.

Jefferies and Davies (1993) identified that a Soil Behaviour Type (SBT) Index, I_c, could represent the SBT zones in the normalized CPT Q_t - F_r chart where, I_c is essentially the radius of concentric circles that define the boundaries of soil type. Robertson and Wride, (1998) modified the definition of I_c to apply to the Robertson (1990) $Q_t – F_r$ chart, as defined by:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5} \quad [1]$$

where:

Normalized cone resistance, $Q_t = (q_t – \sigma_{vo})/\sigma'_{vo}$

Normalized friction ratio, $F_r = [(f_s/(q_t – \sigma_{vo})] 100\%$

q_t = CPT corrected cone resistance

f_s = CPT sleeve friction

σ_{vo} = in-situ total vertical stress

σ'_{vo} = in-situ effective vertical stress

Robertson (2010) further modified equation 1 to apply the I_c concept to the non-normalized SBT chart based on dimensionless cone resistance (q_t/p_a) and friction ratio (R_f).

It is possible to identify the transition from one soil type to another using the rate of change of I_c. When the CPT is in transition from sand to clay the SBT I_c will move from low values in the sand to higher values in the clay. Robertson and Wride (1998) suggested that the approximate boundary between sand-like and clay-like behaviour is around I_c = 2.60. Hence, when the rate of change of I_c is rapid and is crossing the boundary defined by I_c = 2.60, the cone is likely in transition from a sand-like to clay-like soil or vise-versa. Profiles of I_c can provide a simple means to identify and remove these transition zones.

To illustrate this approach an example CPT profile is shown in Figure 2 that was carried out at a site where there were numerous inter-layers of sand and clay. Figure 2 shows the profiles of corrected cone resistance (q_t), friction ratio (R_f), measured pore pressure (u_2), SBT index I_c, and the SBT based on the non-normalized charts by Robertson et al., (1986) and updated by Robertson (2010). The clay/silty clay layers (where I_c > 2.60) are clearly identified on the continuous profile of SBT I_c. The SBT zones are colour coded to aid in the identification (e.g. clay is dark blue when $I_c > 2.6$ and sand is light brown when $I_c < 2.05$, sand-silt mixtures are light-dark green between $2.05 < I_c < 2.60$). The transition zones that were automatically detected based on the rate of change of I_c are identified in red on the SBT I_c profile.

The auto-transition layer detection approach used in the software *CPeT-IT* and *CLiq* (http://www.geologismiki.gr) is illustrated in Figure 3. The approach allows the user to adjust the rate of change of I_c (*Delta I_c ratio*), the number of data points used to determine the rate of change and the range of I_c over which the detection should be carried out.

The example shown in Figures 2 and 3 illustrate that for the CPT profile presented there are potentially 14 transition zones in the 30m profile. Figure 4 shows the example CPTu profile in terms of estimated undrained shear strength ratio (s_u/σ'_{vo}) and OCR for the clay layers both with and without the transition zones. Figure 4 illustrates that, although the automatic detection of transition zones is not perfect, the strength and stress history profiles are improved when the transition zones are mostly removed. For the same example CPTu profile the estimated post earthquake settlements are significantly reduced when the transitions zones are removed. If the transition zones are not removed the post earthquake settlements are conservatively large.

Interpretation of continuous CPT results can be improved if CPT-based software has the ability to both identify and remove transition zones in interlayered deposits. However, the user should have the option to control both the identification and removal of any transition zones so that interpretation can be reviewed both with and without any transition zones to evaluate the impact on any interpretation and/or design.

References:

Ahmadi, M.M., and Robertson, P.K., 2005. Thin layer effects on the CPT qc measurement. Canadian Geotechnical Journal, **42**(9): 1302-1317.

Lunne, T., Robertson, P.K., and Powell, J.J.M., 1997. Cone penetration testing in geotechnical practice. Blackie Academic, EF Spon/Routledge Publ., New York, 1997, 312 pp.

Treadwell, D.D. (1976). "The influence of gravity, prestress, compressibility, and layering on soil resistance to static penetration." PhD thesis, University of California, Berkeley.

Jefferies, M.G., and Davies, M.P., 1993. Use of CPTU to estimate equivalent SPT N_{60}. Geotechnical Testing Journal, ASTM, **16**(4): 458-468.

Robertson, P.K. and Wride, C.E., 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, Ottawa, **35**(3): 442-459.

Robertson, P.K., 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal, **27**(1): 151-158.

Robertson, P.K., 2010. Soil behaviour type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT'10, Huntington Beach, California. www.cpt10.com.

Robertson. P.K., Campanella, R.G., Gillespie, D., and Greig, J., 1986. Use of Piezometer Cone data. In-Situ'86 Use of Ins-itu testing in Geotechnical Engineering, GSP 6 , ASCE, Reston, VA, Specialty Publication, SM 92, pp 1263-1280.

Peter Robertson, Professor Emeritus, University of Alberta, Technical Director, Gregg Drilling & Testing, Inc., Signal Hill, California USA, email: probertson@greggdrilling.com.

The GROUT LINE

Grout Line

Paolo Gazzarrini

Overture

Another issue of the Grout Line, the 24th, and a very interesting article from Dr. Donald Bruce related to rock grouting in dams.

But, before I touch briefly on the content of Dr. Bruce's article I need to express my apologies to Sam Bandimere for a cut/paste mistake in the previous March 2011 issue of the Grout Line. The termination of the article was not correct, a phrase was abruptly cut and another phrase was missing. Here is the "real" and complete text (Sorry Sam, but the computers and human inattention sometimes can provoke this kind of mistake).

ERRATA CORRIGE

I could get on that soap box, but for now will keep that for a future article. In the mean time, I hope there is no engineering firm, contractor, manufacturer or supplier who feels they are too small or insignificant to consider attending any of the industry's educational or conference opportunities. I hope the 2012 ICOG Conference to be held in New Orleans is at the top of your list.

I want to thank the "Grout Line" for giving us a venue for promoting a focus where we govern ourselves by an integrity that promotes the industry as a whole. I, for one, believe our history is a testament to the fact that we are doing a good job of that.

After this due apology I am back to Dr. Bruce's article. I don't think I need to present the author: Dr. Donald Bruce is an international consultant, President of Geosystems, L.P. based in Venetia, PA (dabruce@geosystemsbruce.com). He was past Chair of the ASCE-Geo-Institute Grouting Committee, still a very active member of this organization, and has been an instructor at the Colorado School of Mines in the Annual Grouting Course since 1988. He is the co-author of the textbooks "Dam Foundation Grouting" (2007), "Ground Control and Improvement" (1994), and the author of the upcoming book, "Specialty Construction Techniques for Dam and Levee Remediation: The U.S. Technology Review."

The article expresses a very clear point of view, arguable –in my opinion- in some drastic judgments specifically about the GIN Method, but definitely very interesting, and it is my hope that it will open some constructive discussions that can benefit our grouting industry. I received it quite late and I will comment, personally, in the next September issue. If you have an opinion about it please send me your comments. An update about the 2012 Grouting Conference: everything is proceeding on schedule. All the papers have been reviewed and we are expecting a very busy week next year in New Orleans. Write to: Paolo Gazzarrini, fax 604-913 0106 or paolo@paologaz.com, paologaz@shaw.ca or paolo@groutline.com. Or tweet me @groutline.

Ciao!

Rock Grouting for Dams and the Need to Fight Regressive Thinking

Dr. Donald A. Bruce, C.Eng.

Abstract

It is generally recognized, both nationally and internationally, that rock grouting theory and practice in North America has undergone a most positive revolution during the last decade or so. Key elements of this progress have included the development and use of suites of balanced, stable High Mobility Grouts (HMG); increasing use of Low Mobility Grouts (LMG); new overburden and rock drilling methods; computer monitoring control and analysis; and the use of Apparent Lugeon Theory and Lugeon testing to assure proper stage refusals and low residual permeabilities, respectively. These concepts have been most strongly implemented on major Federal dam remediation projects. Also, certain

consultants are using them on smaller, non-Federal projects.

However, the author has noted over the past few years a distinctly retrogressive faction in the grouting industry which, if left unchallenged, will undo much of the advantages gained over the last decade. Examples include a reversion to the use of highly unstable HMG's as engineers confuse "thin" and high water content. Perhaps more concerning is the re-emergence in certain circles of the thirty-year-old GIN Method (Grouting Intensity Number). This method was devised with the laudable goal of trying to assure a certain basic standard of care in grouting projects in countries of a lesser degree of resource and sophistication.

In this paper, the author urges against the regression in U.S. grouting practice, which is in danger of occurring due to a relapse into old, unsatisfactory habits, and a "rediscovery" of outdated and inappropriate methodologies. The U.S. grouting industry today is ranked amongst the most active and effective in the world, and this level of approbation should be guarded and cultivated, not let slide.

Introduction

Rock grouting for dam foundations has been carried out in the U.S. since at least 1893 when the limestone bedrock of a dam in the New Croton Project, NY, was treated with cement grout (Franklin and Dusseault, 1989). Opinions differ on the method of injections (Glossop, 1961, Littlejohn , 2003), although other reports (Verfel, 1989) strongly suggest that U.S. grouting procedures had made "a good start."

For the best part of the following hundred years, the intense history of dam grouting in the U.S. is, to some extent, a picture of objectives not fully achieved, innovative procedures and insightful ideas inconsistently implemented, and a number of questionable practices unthinkingly perpetuated. During the last 15 years, however, in many — but not all — parts of our practice, there has been a radical change in our concepts and in our approaches to such work. Partly drawing from knowledge made available in the U.S. by European specialists, for example at the seminal grouting conferences hosted in New Orleans in 1982, 1992 and 2003, and partly by the very challenging problems posed by the need to construct remedial grout curtains in our own dams, especially on karst, there has been a technological revolution in dam grouting practices in the U.S. This revolution has greatly benefited the owners of these dams, and dams themselves, and — by association — the grouting profession at large.

However, the proven advantages and successes of this uniquely tailored advance have not yet everywhere been recognized, and have not always been upheld and consistently defended. We therefore find that in some regions, or in certain organizations — or most sadly in certain sections of certain organizations — rock grouting is still being specified in the terms of 50 years ago. Equally, there are increasing numbers of projects being specified and run according to "new concepts" which, in reality, are new only to the designers and represent a retrogressive step of almost 30 years.

In the following sections, the old, the new and the retrogressive concepts of rock fissure grouting are presented to provide a platform for logically arguing against the old and the retrogressive ways of approaching work of this type. Given the relatively high volume of dam grouting — especially for remedial applications — being conducted today, we have now arrived at a particularly important time to have this debate.

Historical Concepts ("The Old")

There is a trove of published information to be found on this subject, including the Proceedings from the New Orleans Grouting Conference in 1982, the "Foundations for Dams" Conference (1974) and textbooks by Houlsby (1990), and Weaver (1991) in particular. Even more important are the unpublished reports, memoranda and manuals produced on a project-specific basis, or by companies or governmental organizations. These had special gravitas because their authors strongly influenced the next generation of grouting engineers while they, themselves, were elevated to the position of "consultants" on other projects in different governances.

Bearing in mind the unprecedented level of activity in those years in new dam grouting, as well as the national puritanism towards "low bid" contracting, specifications were highly prescriptive and restrictive. Such prescriptions did nothing to stimulate innovation since the contractor was reduced to the status of the cheapest purveyor of labor, equipment and materials, while the goal of the owners' inspectors was to ensure that the specifications were enforced to the letter, via "hole by hole" direction of the grouting activities.

By the way of illustration, in 1974, Polatty was invited to give an overview of U.S. Dam Grouting Practices: "In preparing this paper, I requested copies of current specifications for foundation grouting from several Corps of Engineers districts, the TVA and the Bureau of Reclamation. In comparing these current specifications with copies of specifications that I had in my file that are 30 years old, plus my observations and experience, I concluded that we in the United States have not, in general, changed any of our approaches on grouting. AND THIS IS GOOD" (emphasis added). Interestingly, he then went on to list "difficulty in having sufficient flexibility in the field to make necessary changes to ensure a good grouting job" as a problem. What a surprise!

As a consequence, several important historical paradigms became embedded in our national practice as late as the 1980's. These include:

- The drilling of vertical holes, to a target depth (as opposed to stratigraphic horizon). The only common exception (e.g., Albritton, 1982) would be the concept of inclining the curtain upstream, so as to physically distance it from the downstream drains.
- The use of rotary drilling (often just coring) since in the early days of the 20th century since only such drills could use water flush. Percus-

sion drilling was then synonymous with the use of air flush, which many (but not all) did recognize as detrimental to fissure cleanliness and amenability to grout. (The age old debate about rotary versus percussion drilling as being more suitable for grout holes was wrongly focused: it should have been water versus air.)

- The concept of a "one row curtain," except notably under the cores of embankment dams, where even then the shallowest possible excuse was taken to revert to one row.
- The use of relatively low grout pressures, resulting from the recurrent specification to provide "constant" pressures which therefore meant the use of progressive cavity pumps ("Moynos") as opposed to higher pressure piston or ram pumps.
- The use of "thin" grouts (with excessive water:cement ratios often well in excess of 10 by weight – although typically mixes were measured by volume). Such mixes of course were easy to pump due to their low apparent viscosity, but naturally had extremely high bleed values and horrible pressure filtration resistance. These mixes were allied with a fundamental distrust/unawareness of the benefits of additives (except for calcium chloride in "taker" situations) although, latterly, the use of bentonite was entertained and ongoing though somewhat misguided experimentation with superplasticizers was conducted in certain quarters.
- Curtains were grouted until a certain cement refusal was obtained (e.g., 1 bag per foot) as opposed to a measured residual permeability. This is, however, a charitable view: often the grouting was discontinued when the budget was expended and, in the aftermath when the underseepage became of alarming quantities, the cry was made that "the grouting didn't work!" The general result (Weaver and Bruce, 2007) of these deficiencies was either a) a poor travel of grout in the ground, leading to the drilling of families of higher order holes at ridiculously close centers (e.g., 1 foot at Chickamauga Dam, TN), or b) uncontrollable flow of "thin" grouts into karstic voids or similar major features.

It is somewhat of a testament to the enlightened, the lucky, and the meticulous that so many of the curtains constructed in the period from the 1920's to the early 1980's in particular appear to have actually functioned adequately given the restrictions, the misconceptions and the prescriptions. Uncharitable views would have it that such curtains may not have been needed at all, from a dam performance or safety viewpoint, and that the curtain was inserted by rote and by paradigm. On the other hand, the fact that so many of our dams have now been remediated, or are facing remediation as a result of an ineffective, incomplete and/or deteriorating grout curtain, does lead us back to the inescapable fact that the "old ways" in retrospect contained major flaws in their workings. One definition of the word "insanity" is to continue to do the same thing even when it has been repeatedly proved to fail or to be wrong. To persist with, or revert to, the "old" ways of grouting dam foundations is an example of this definition.

Current Principles ("The New")

There had arrived in the North American scene by the mid-1990's a potent mixture of knowledge and opportunity. As arguably first articulated at a Grouting Seminar in Toronto, ON in 1989, but certainly emphasized to the cogniscenti in New Orleans in 1992, the world of dam grouting in North America had begun to change dramatically. This statement is made with all due recognition of Dr. Wally Baker who, some years before, had instigated an advance into new technical fields, but an advance which proved economically unsustainable in the face of prevalent contracting and procurement vehicles of the time.

Of particular significance was a paper by DePaoli et al. (1992) which, in a deceptively understated way, explained quite clearly the critical control and importance of pressure filtration coefficient over the effective travel of grouts into fissures, and hence their efficiency in generating low and durable residual rock mass permeability. As described in Weaver and Bruce (2007), pressure filtration can be conceived as follows:

"The injection of particulate grouts into small apertures is similar to pressing the grout against a filter material: depending on the formulation of the grout, water can be expelled from the grout in motion, leading to the development of cementitious filter cake at the borehole wall. With more time, the cake blocks off the entrance to the aperture and so renders the aperture inaccessible to further injection via that avenue. This tendency of the grout to lose water during injection is quantified by the term *pressure filtration coefficient* (Kpf)…"

"To enhance the penetrability of a grout, a low-pressure filtration coefficient that minimizes the increase in apparent viscosity (Figure 1) is required. The general relationship between the two vital parameters of cohesion and pressure filtration coefficient is shown

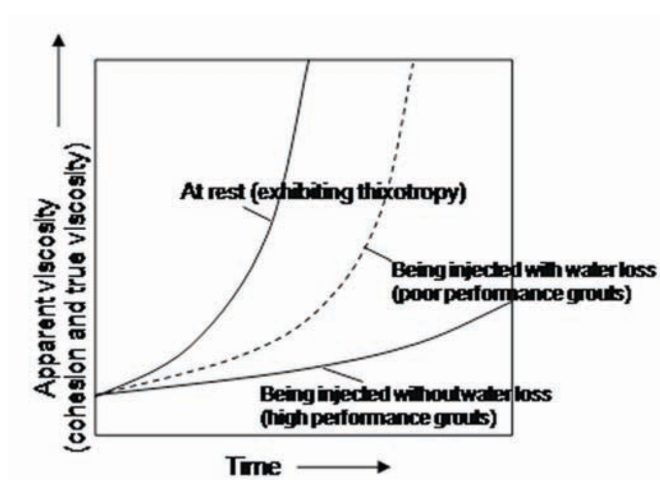


Figure 1. Rheological behavior of typical Binghamian fluids (modified after Mongilardi and Tornaghi, 1986).

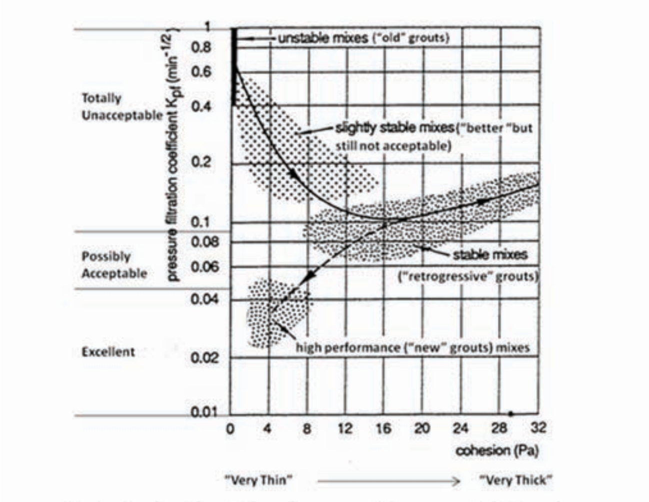


Figure 2. Historical path of development from unstable mixes to contemporary balanced multi-component mixes (modified after De Paoli et al., 1992).

Photograph 1. (Courtesy of ACT and Gannett Fleming, Inc.).

in Figure 2. Whereas cohesion was traditionally minimized in simple cement–water grouts by using extremely high w:c ratios (Albritton 1982), such mixes have high Kpf values, which severely curtail their penetrability. However, by using much lower water contents (typically less than 1.5 weight by volume) and combinations of stabilizing and plasticizing admixtures and additives (including bentonite, silica fume, and Welan Gum), grouts of low viscosity (less than 60 seconds Marsh), low cohesion, minimal bleed, and excellent Kpf values (less than 0.02 min–1/2) can be produced. DePaoli et al. (1992) found that even under moderate injection pressures, such balanced, stabilized grouts provided enhanced penetrability and performance via the following:

- an increased radius of travel;
- a more efficient sealing ability as a result of the improved penetrability and the lower permeability of the mix;
- a high volumetric yield, with uniformly filled voids; and
- a higher erosion resistance because of improved mechanical strength for a given cement content."

Of course, it must be acknowledged that other factors will impact curtain effectiveness, but never in the U.S. literature before 1992 was the significance (or even concept) of "pressure filtration" mentioned in conjunction with rock grouting.

It is only fair to separate from the comparison between "old" and "new" those elements which are, by invention and technology, the exclusive privilege of the "new." Much has been written and rightly so, about the tremendously beneficial effect that the use of computer-based systems have had on the collection, processing, interpretation and display of data from the field (Dreese et al., 2003). No reputable grouting project of any significant scale or importance does now not have such a capability, feeding news back into a central "mission control" (Photograph 1), and back into the Project Executive's desk in head office, as well. The best of these systems can now integrate all the drilling and water testing data, as well as the grouting data, to compliment and compare with the historical site investigation data (and original grouting information) which may be available on any particular project. Given the power of this knowledge, curtains can be constructed to engineered standards with a degree of reliability and confidence which was unthinkable under old regimes.

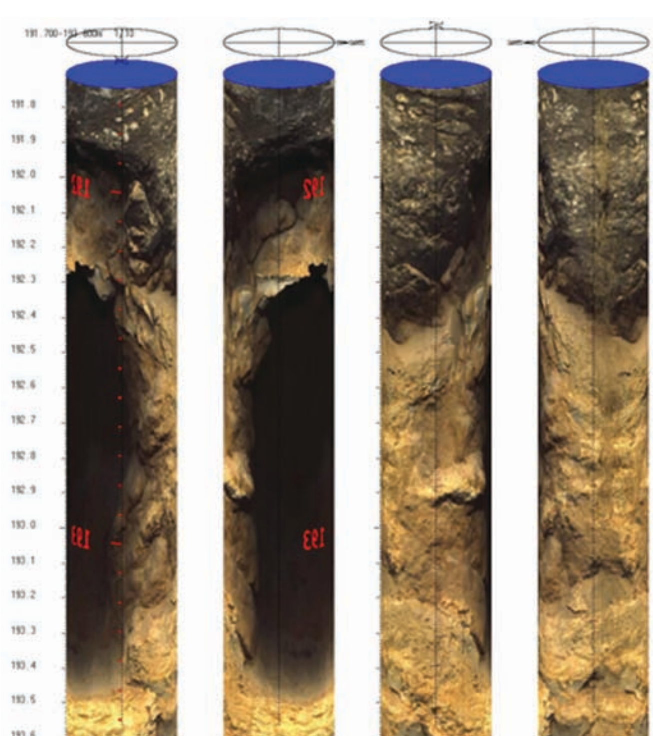
Another child of the new age is the Optical or Accoustic Televiewer, an extremely acute and reliable instrument which basically provides a "flat core" of a preexisting hole (Photograph 2). With this capability, the borehole wall conditions of drill holes — formed "destructively" without the expense of core drilling — can be closely scrutinized, and compared with results from permeability tests and grout injections. This is an extremely important diagnostic tool, and represents a compatibility far beyond the grainy, boring images hitherto provided by down-the-hole video cameras.

Returning to a comparison of "old" and "new" concepts, the fundamental change in attitudes towards mix designs and mix properties has already been discussed: it is one absolutely vital component in the revolution. However, even today, the author finds specifications — or worse, projects — where the grout mix design comprises three components at best, and mixes are changed from "thin" to "thick" by changing from water:cement ratios of 3:1 to 0.8:1, or 0.6:1 in the case of "gulpers." This is simply inexcusable and not acceptable given the state of knowledge which currently exists and is freely available on this subject.

Other areas of important distinction in contemporary grout curtain design and construction may be summarized as follows:

- **Curtain Geometry:** Curtains must have, as a minimum, 2 rows of holes, which extend, wherever feasible technically, into a confining layer. They are not simply installed to a target depth below ground surface. Also, the holes in each row are inclined say 15º off vertical. The inclination of each row of holes is in the opposite direction, thereby producing a "criss cross" effect, assured to intercept all fissure sets, especially those vertically oriented. The zone between these "outer rows," typically about 10 feet wide, is then available for additional "tightening" holes, perhaps using special or different grouting materials, and for drilling and testing Verification Borings which are installed to demonstrate the residual permeability achieved by the curtain.

- **Residual Permeability:** The purpose of a grout curtain is to stop water flowing through the rock mass. Therefore, its acceptability as an engineered structure must be verified by measuring its residual permeability — to water, not some arbitrary limiting grout take. (As described above, an inappropriate grout will have premature refusal in certain fissures, while not reducing the permeability of the ground further away.) This test is best done in cored (or Optilogged) holes, using multipressure Lugeon Tests as first described by Houlsby (1976).
- **Declaring the Target Residual Permeability:** Residual permeability is the goal which must be declared as part of the design by the Engineer and which therefore must be satisfied by the Contractor. A grout curtain truly now is a "Quantitatively Engineered" structure (Wilson and Dreese, 2003), created by real-time control of subsurface construction processes. This "measure of success" will vary from project to project, as articulated by, for example, Houlsby (1990), but is vital to declare and essential to satisfy.
- **Stage Refusal**: Each and every stage should now be brought to a virtually total refusal. When viewing the grouting process on the computer monitor, this means an Apparent Lugeon Value of practically zero for each stage (i.e., the (stable) grout is used as a test fluid in the same way as water is). In reality, this means that the stage in question is consuming grout at less than 0.1 gpm over a period of, say, 5 minutes, at target pressure. More lax refusal criteria will result in incompletely and inefficiently grouted stages, and so higher than desirable residual permeabilities in the rock mass.
- **Drilling Methods and Concepts:** Water is the drilling and flushing medium of choice in rock masses. Whether the drilling is done by percussive methods (top hole, or water-powered down-the-hole hammer) or rotary methods (which tend now to be less competitive and have greater deviations) is technically immaterial. Also, the development of commercially viable rotary-sonic systems (Bruce and Davis, 2005) has provided a method which has entirely satisfied federal regulations (USACE, 1997) for drilling through existing embankment dams without fear of hydro or pneumatic fracture. In this regard, it is also the case that innovative contractors can devise other conforming overburden drilling systems which are equally protective of embankment fills (Photograph 3).



Photograph 2. (Courtesy of ACT and Gannett Fleming, Inc.).

In all drilling operations, the recording of drilling parameters (e.g., rate of penetration, flush characteristics, torque and so on) has been regularized by developing automatic recorders as opposed to relying on drillers or junior field engineers: the overall rise in the quality and usefulness of these data has been predictably spectacular.

- **Specifications and Contractor Procurement Processes**: Specifications are no longer so prescriptive ("yes: we do need the head of the contractor as well as his arms") and so all contracts are not let on the low bid basis, although to do otherwise is still not permissible for many organizations, especially in the public sector. Grouting contractors are being hired, correctly, based on their skills and experience and not just their capability of calculating a low price. There is absolutely no doubt that this "Best Value" approach has raised technical standards across the board and has, interestingly, honed the competitive instincts of all competent contractors: all this is to the inestimable benefit of the projects themselves. Further insight on specifications is provided in Bruce and Dreese (2010).

Retrogressive Principles (i.e., "The Retrogressive")

The fact is that certain engineers in North America have become involved in projects where a certain expertise in curtain grouting is needed and where, for commercial reasons, they have chosen to go their own way, and/or to "reinvent the wheel." The uncomfortable truth is that they are either not aware of the "new" approach, or do not have the technical background to be able to differentiate its value in comparison with older principles advocated by "old friends" in the industry who are in fact typically not grouting engineers by practice. A classic example of this is the sporadic emergence of the use of GIN Theory as the guiding principle for curtain grouting on certain U.S. projects.

Dr. Giovanni Lombardi is a Swiss dam engineering expert who is an extremely influential figure, especially in developing countries. His long association with Dr. Don Deere, particularly in projects involving curtain grouting on South American dams, included the development in the 1980's of "GIN Theory" (the GIN component referring to "Grouting Intensity Number"). This was laudably developed to assure for the client a certain standard of care and quality would be achieved on projects which were in remote areas and/or were to be built by contractors with (somewhat) limited experience and expertise.

Together by 1993 they had articulated an approach to grouting that takes into account the specific energy expended in the injection process. Their approach assumes that, for any given interval, the energy expended is approximately equal to the product of the final pressure (p) and the volume (V) of grout that is injected. The numerical value of this product is called the grouting intensity number, or GIN. Depending on the units used, this number may be expressed in bar-liters per meter. They recommended taking into account site-specific factors, including the ultimate reservoir head, the characteristics of the bedrock discontinuities, stratification, weak zones, weathering, and in situ state of stress in selecting a GIN number that — in conjunction with limiting values of volume and pressure — is to be used for easily grouted fissures as well as for finer fissures. They reasoned that because the pressure decreases quite rapidly as the grout moves away from the borehole in tight fissures, the total uplift pressure even at high injection pressures will as a rule be much lower than the overburden weight, except in the uppermost 5 to 10 m of the foundation. On that basis, they indicated that a limiting pressure as high as 50 bars might be appropriate if high-intensity grouting were desired. However, for most conditions, they recommended using a limiting pressure of 30 bars and a limiting volume of 200 L/m.

Perhaps anticipating objections to the grout volume limitation imposed by the GIN rule, Lombardi (2003) stated that the nominal limitation could actually be treated as a decision point rather than as an absolute, rigid stopping point. He suggested that the decision might be one of the following:

- Continue injection of grout.
- Terminate injection of grout.
- Temporarily stop grouting and resume injection after a period of time.
- Abandon the hole and drill another nearby.
- Add a product, for example, an anti-washout agent, to the grout mix, or take some other appropriate measure

It would appear that application of the GIN principle entails use of a single "moderately thick" superplasticized stable grout, defined in this case as a grout with less than 5% bleed after 2 hours, throughout the injection process. This grout is injected at a steady low to moderate rate, allowing the pressure to build up gradually as the grout penetrates farther into the foundation rock mass. Real-time monitoring of a series of relationships or parameters by computer graphics is required. These relationships and parameters include curves of pressure versus time, grout flow rate versus time, total injected volume versus time, and the derivative curve of flow rate divided by pressure versus time.

Lombardi and Deere (1993) stated that the GIN principle had been used in construction of grout curtains for dams in Turkey, Mexico, Argentina, Austria, Switzerland, and Ecuador. However, Ewert (2003) vociferously pointed out that application of the GIN principle in certain geologic conditions and in some rock types may be inappropriate, especially if the grouting program is in the hands of inexperienced personnel. His adverse opinions regarding the GIN principle included the following:

- The maximum pressures proposed by the principle are too high for most rock types, causing hydrofracturing and unnecessarily large grout takes.
- The maximum volumes allowed by the principle when grouting at low pressures are inadequate to ensure complete filling of wider open joints.

Photograph 3.

That particular, and memorable, technical session in New Orleans in 2003 continued at a rapid pace for much longer than the organizers had intended, reflecting much credit in both protagonists. The opinion of the author is as follows:

- Hydraulic fracturing (and, for that matter, fracture dilation or surface displacement) can readily and quickly be recognized by competent, experienced personnel using modern real-time monitoring equipment and procedures. Injection pressures can then be reduced, and injection can be slowed or stopped as appropriate before excessive volumes of grout are injected.
- On the second point, although adjusting the rheology of the grout rather than halting injection to limit grout travel after some prescribed maximum volume has been injected, is favorable application of conservative curtain closure criteria and procedures would, in most if not all cases, provide additional opportunities to complete the filling of wider open joints.

In retrospect, had the esteemed Dr. Lombardi strongly and widely promoted his Theory in the U.S. during the time of the U.S. grouting industry “fin de siècle” (i.e., the late 1980’s), then it is highly probable that the entire North American continent, in addition to South America, would have had a different grouting direction. Instead, this flare from Europe has fallen between the two stools of the U.S. grouting practice, one anchored in the early 1920’s, the other springing from the revolution of the mid-1990’s. In summary, GIN Theory most probably has worked well and was an excellent option in the grouting interregnum in developing countries during the latter decades of the 20th Century. However, the approaches developed in North America over the last 15 years have been verified to give truly exceptional, compliant and consistent results, using means and methods which are site-specific. In effect, there was a "GIN window" in the U.S. between 1985 and 1995 during which GIN Theory could have become predominant,but did not, and to now to promote it in the U.S. is regressive.

Conclusion

Current U.S. dam curtain grouting practice for seepage control has evolved during the last 15 years or so to a level that it can assure a responsive and effective solution to any project-specific challenge, be it a remedial application, or a new dam curtain. For the benefit of the industry, it is essential that two tasks are implemented. The first is to eradicate the “old ways”: this in itself is a matter of technical education, although the quality, intensity and consistency of the education need to be pursued with constancy and vigor. The second is to be on guard against regression, which is typified by adoption of concepts which were popular decades ago in other countries, but for different reasons, did not arrive in the U.S.

For the first time in our dam grouting history, North America has a current approach and a track record which is without equal in the world. This is partly due to the severity of the challenges we face, but also is a result of a typically uniquely North American mélange of concepts and resources. The recent record speaks for itself with excellent results having been achieved on remedial grouting projects at many USACE DSAC-1 projects in particular. While we should and will remain receptive to new developments, we must not allow the industry to give up the successes of the last decade.

References

Albritton, J.A. (1982). “Cement Grouting Practices U.S. Army Corps of Engineers.” American Society of Civil Engineers, Geotechnical Engineering Specialty Conference on Grouting, New Orleans, February 10-12, pp. 264-278.

Bruce, D.A., J.P. Davis. (2005). “Drilling through Embankments: The State of Practice,” USSD 2005 Conference, Salt Lake City, UT, June 6-10, 12 p.

Bruce, D.A. and T.L. Dreese. (2010). “Specifications for Rock Mass Grouting,” ASDSO Dam Safety Conference, September 19-23, Seattle, WA, 12 p.

DePaoli, B., B. Bosco, R. Granata, and D.A. Bruce. (1992). “Fundamental Observations on Cement Based Grouts (1) : Traditional Materials.” Proc. ASCE Conference, “Grouting, Soil Improvement and Geosynthetics,” New Orleans, LA, February 25-28, 2 Volumes, pp. 474-485.

Dreese, T.L., D.B. Wilson, D.M. Heenan, and J. Cockburn. (2003). “State of the Art in Computer Monitoring and Analysis of Grouting.” Grouting and Ground Treatment, Proceedings of the Third International Conference, Geotechnical Special Publication No. 120, Ed. L.F. Johnsen, D.A. Bruce, and M.J. Byle, American Society of Civil Engineers, pp. 1440-1453.

Ewert, F.K. (2003). “Discussion of Rock Type Related Criteria for Curtain Grouting.” Proceedings of the Third International Conference on Grouting and Ground Improvement, ASCE Special Publication No. 120.

Franklin, J.A. and M.B. Dusseault. (1989). “Rock Engineering.” McGraw-Hill, New York, March, 610 p.

Foundation for Dams. (1974). ASCE Proceedings, Engineering Foundation Conference, Asilomar Conference Gorunds, Pacific Grove, CA, March 17-21, 472 p.

Glossop, R. (1961). “The Invention and Development of Injection Processes, Part 2, 1850–1960.” Géotechnique, 11, 4, December, 255–279.

Houlsby, A.C. (1976). “Routine Interpretation of the Lugeon Water-Test.” Quarterly Journal of Engineering Geology, 9(4), 303-313.

Houlsby, A.C. (1990). “Construction and Design of Cement Grouting.” John Wiley and Sons, 442 p.

Littlejohn, G.S. (2003). “The Development of Practice in Permeation and Compensation Grouting: A Historical Review (1802 – 2002) Part 1 Permeation Grouting.” Grouting and Ground Treatment, Proceedings of the Third International Conference, Geotechnical Special Publication No. 120, Ed. L.F. Johnsen, D.A. Bruce, and M.J. Byle, American Society of Civil Engineers, pp. 50-99.

Lombardi, G. and D.U. Deere. (1993). “Grouting Design and Control Using the GIN Principle.” International Water Power and Dam Construction. 45 (6), 15-22.

Mongilardi, E. and R. Tornaghi. (1986). “Construction of Large Underground Openings and Use of Grouts,” Proceedings of the International Conference on Deep Foundations, September, Beijing, The Deep Foundations Institute, 19 p.

New Orleans Grouting Conference. (1982). Proceedings, ASCE Geotechnical Engineering Specialty Conference on Grouting, New Orleans, February 10-12.

New Orleans Grouting Conference. (1992). Proceedings, ASCE Conference, Grouting, Soil Improvement and Geosynthetics, New Orleans, LA, February 25-28.

New Orleans Grouting Conference. (2003). Grouting and Ground Treatment, Proceedings of the Third International Conference, Geotechnical Special Publication No. 120. Edited by L.F. Johnsen, D.A. Bruce, and M.J. Byle, American Society of Civil Engineers, New Orleans, LA, February 10-12, 1,663 p.

Polatty (1974). ASCE Proceedings, Engineering Foundation Conference, Asilomar Conference Gorunds, Pacific Grove, CA, March 17-21, 472 p.

U.S. Army Corps of Engineers (1997). “Engineering and Design Procedures for Drilling in Earth Embankments,” CECW-EG, Report No. 1110-1-1807, September 30.

Verfel, J. (1989). “Rock Grouting and Diaphragm Wall Construction,” Elsevier, New York.

Weaver, K.D. (1991). “Dam Foundation Grouting.” ASCE, New York.

Weaver, K.D. and D.A. Bruce. (2007). “Dam Foundation Grouting, Revised and Expanded Edition,” American Society of Civil Engineers, ASCE Press, New York, 504 p.

Wilson, D.B.. and T.L. Dreese. (2003). “Quantitatively Engineered Grout Curtains,” Grouting and Ground Treatment, Proceedings of the Conference sponsored by the Geotechnical Engineering Division of the American Society of Civil Engineers, New Orleans, LA, February 10-12, pp. 881-892.

Dr. Donald A. Bruce, President, Geosystems, L.P., P.O. Box 237, Venetia, PA 15367, U.S.A.; Phone: (724) 942-0570; Fax: (724) 942-1911; Email: dabruce@geosystemsbruce.com

WASTE GEO Technics

From the Moon to the Oil Sands

Vivian Giang

Land reclamation is a top research priority for the oil sands industry. World-leading geotechnical engineering experts have been enlisted to tackle the challenging tailings ponds to ensure the sustainable development of Alberta's oil sands. How far do you go to find these experts? The moon.

In 1969, at the age of 25 and as a four-month veteran of NASA, Dr. W. David Carrier III made one of the best predictions of his life.

"We were trying to predict the density of lunar soil versus its depth," says Carrier, founder and Director of the Lunar Geotechnical Institute, a non-profit research corporation based in Florida. "I briefed the Apollo 11 astronauts on what to expect when they stepped on the moon. They had been told many things… like they would sink out of sight into the lunar dust or that the dust would cling electrostatically to their suits in a thick layer, making it impossible to see," he chuckles. "Luckily, my prediction was right."

With limited data available about the moon at the time, Carrier believed that lunar soil was denser than the consistency of "sifted flour" which most people speculated at the time. In a 2002 interview with *Air & Space Magazine*, Neil Armstrong commented, "We found the predictions of Dr. David Carrier and the Soil Mechanics team to be more persuasive than [others']."

Carrier instructed Armstrong and Buzz Aldrin to photograph the undisturbed ground and then their bootprints. As he suspected, the astronauts' boots sunk about one centimeter into meteorite-disturbed soil and planted firmly into the moon's dense surface. Incidentally, the bootprint photograph became one of the most famous icons in American history. However, the photograph served a greater purpose: it gave Carrier insight into the trafficability of lunar soil and was instrumental in understanding the moon's soil mechanics on a larger scale.

How does all of this relate to oil sands tailings?

"There's a connection between the lunar experiments and my tailings work: predicting the density of the soil versus its depth," says Carrier.

"That is one of the characteristics of David," says Dr. J.D. Scott, Professor Emeritus at the University of Alberta, who has worked with Carrier on consolidation testing. "He has the big picture in mind and isn't narrow in outlook. He has a broad knowledge of the geotechnical field, which allows him to bridge his lunar work to tailings."

Carrier left NASA in 1973 and worked as an engineer for over 20 years prior to establishing his consulting firm Argila Enterprises, Inc. ("argila" is the Portuguese word for "clay"). As an engineer in the late 1970s, Carrier began investigating the disposal of mineral waste materials and the design of retention areas for phosphatic clays in Florida. His densification (or dewatering) work included theoretical, applied, and laboratory research. During a meeting where Carrier discussed advances in consolidation techniques, Roger Wiley of Syncrude Canada Ltd. saw the possibility of applying Carrier's research to the oil sands industry and immediately invited Carrier to visit Northern Alberta. Carrier worked extensively with Wiley, Bill Shaw, and Bruce Friesen during that time and more recently with Rick Lahaie and Geoff Halferdahl.

"David brings a wide range of experience to bear on the consolidation of fluid fine tails, and it's that worldwide experience and special ability to operate empirically that makes him

Carrier working with one of the Apollo astronauts on a drilling experiment at the Kennedy Space Center in Florida circa 1970.

Carrier working at a test pit with bauxite tailings in the Amazon jungle of Brazil.

successful," says Lahaie. "We have a large data set overlaid by many years' worth of information. David can manage and work amongst all the data and simplify it into useful metrics. On top of that, he has a lot of practical knowledge. He marries the two in a very utilitarian way."

Carrier is currently working on accelerated dewatering methods at Syncrude's Mildred Lake project near Fort McMurray with Jack Seto of BGC Engineering Inc. and Murray Fredlund of SoilVision Systems Ltd. Carrier, who conducted notable rim ditching research in Florida, believes Terzaghi's effective stress principle can be applied to the soft, yoghurt-like fine tailings. "We needed to change the framework and see this as a mechanistic process rather than a chemical one," says Carrier.

The combination of rim ditching on the perimeter of the deposit with evaporation on the top of the deposit will increase the effective stress, creating a landform with greater trafficable strength. According to Carrier, the key is to drain the water off so that it can evaporate and crack. Water draining and surface cracking will lead to the lowering of the water table, which applies a larger load on the layers underneath. "You leave it for a year and then come back to cut a bit deeper around the perimeter. Then leave it again for year. You keep doing this until the soil is dense enough that you can reclaim it and pour sand on it or plant vegetation on it," he says.

Though the methodology is relatively simple, Carrier says the primary concern is getting the produced tailings into a manageable volume. Then, the next question is how to make it stronger. Carrier believes all the technologies that the industry is looking at now, such as centrifuging and thin-lift deposition, will result in denser, more manageable, more environmentally friendly and sustainable material. Yet he is quick to point out that dewatering is not without its challenges. "The more densification we do, the more water we produce within the system," he says. "The long-term challenge for the industry is reducing the volume of water stored and ensuring the reclamation of it."

Carrier's experiences prove that the sky is certainly not the limit when it comes to tackling tailings issues. The rim ditching research done by Carrier and his Florida colleagues has resulted in the reclamation of 50,000 hectares (500 km^2) of phosphatic clay ponds. Currently there are 426 km^2 of disturbed land in Alberta's Athabasca region. In an effort to conduct collaborative research on the early reclamation of tailings ponds in Alberta, the Canadian Oil Sands Network for Research and Development (CONRAD)

Digging Towards Success

David Carrier is standing on crusted oil sand flocculated fluid fine tailings after one year of dewatering at the Syncrude Canada Ltd. Flocculated Fluid Fine Tailings Perimeter Ditch Dewatering Pilot Project at its Base Lease Operations north of Fort McMurray, Alberta.

Carrier is standing on the edge of flocculated fluid fine tailings (a "silty clay" flocculated with polymers) that have been deposited in an impoundment approximately 80 m by 80 m by 10 m deep and is dewatering mostly by surface evaporation and water flow to a progressively deepened perimeter ditch that is connected to a drain. The perimeter ditch concept is patterned after similar work that Carrier has been involved in Florida with dewatering clays from the phosphate industry.

Carrier is wearing a life jacket and is being very careful not to break through the crust that he is standing on and into underlying fluid-like tailings below the crust that has not yet dewatered. Where he is standing, there is approximately a 20 cm crust on about 1 m depth of fluid fine tailings.

Note the surface-cracking network that has developed due to shrinkage of the material as it dewaters. The ponded water behind him in the center of the Pilot deposit is from recent rains. This water has not yet either evaporated or found its way through the surface-cracking network and perimeter ditch to a drain where it is removed from the Pilot.

The floating dock way and geotechnical instrumentation poles (piezometers, thermistors, and settlement gauges) behind Carrier are in the center of the Pilot. The fence around the perimeter of the Pilot keeps animals and people from wandering into the Pilot.

— Geoff Halferdahl, Syncrude Canada Ltd.

announced the establishment of the Oil Sands Tailings Consortium (OSTC) in December 2010. The consortium consists of oil sands companies including Canadian Natural Resources Ltd., Imperial Oil, Shell Canada, Suncor, Syncrude Canada Ltd., Teck Resources, and Total E&P. Fortunately, these companies no longer have to go to the moon to find superstar researchers like Carrier to work on the tailings ponds… only to Florida, where Carrier currently resides.

Acknowledgements

Thank you to Dr. Carrier and Syncrude Canada Ltd. for supplying photographs for this article.

References

Bourque, J. (2002). “Shooting the moon.” *Air & Space Magazine*. 01 May 2002. Retrieved from http://www.airspacemag.com/space-exploration/shooting-moon.html

Carrier, W.D. (2001). “Lessons Learned – So Far.” Geotechnical News, Vol. 19, No. 3, pp. 45, 55.

Vivian Giang is the Communications & Grant Strategist at the University of Alberta Geotechnical Centre, Department of Civil & Environmental Engineering, Edmonton, Alberta, Canada. Tel: 780-492-1436
Fax: 780-492-0249
email: viviang@ualberta.ca

News from Geoengineer.org – A Center for Information Dissemination on Geoengineering

1,000,000 visitors in 2010 and counting!

We announce that our Center’s website traffic exceeded 1,000,000 visitors in 2010! In 2010 we had a total of 1,000,592 visitors and 12,485,927 page views!! The Center’s statistics for 2010 are available online. Thank you for your support and please keep sending us feedback about your professional needs and how our Information Center can help. Visit Geoengineer.org at http://www.geoengineer.org.

Geotechnical Literature Finder (GEOLIFI): view the 10 most popular papers of the last 3 months.

In celebration of this first year of GEOLIFI, has added a list of the 10 most viewed papers of the past 3 months. Visit the database at http://geolifi.geoengineer.org to find out more. GEOLIFI aims to become the most comprehensive online geotechnical database. If you have access to geotechnical publications and would like to include them in GEOLIFI, please contact us.

Online Geoengineering Library: our electronic collection now exceeds 2,300 free publications!

The Online Geoengineering Library has recently enhanced its collection with the addition of more than 200 new publications. Browse our database of 2,315 documents at http://www.geoengineer.org/onlinelibrary/.

International Journal of Geoengineering Case Histories (IJGCH): New issue published.

Issue#1 of Volume#2 of the IJGCH has been circulated. Papers of the current issue as well as past ones are available online for free. Just visit the journal’s website at http://casehistories.geoengineer.org for further reading and downloading. The IJGCH is always looking for papers on well-documented case histories with emphasis on observations and data collected before, during and after project construction. You may submit your paper for consideration and possible publication at any time.

Prof. Ralph B. Peck’s Legacy Website: More photos have been added!

New photos have been added to the gallery of the Prof. Peck’s Legacy Website from the 4th International Conference on Case Histories in Geotechnical Engineering, March 9-12, 1998. If you would like to contribute to the enhancement of this website please share any information or other resources you may have with us. Prof. Ralph B. Peck’s Legacy Website can be found at: http://peck.geoengineer.org.

Geoengineer.org Newsletter: the easiest way for you to stay up-to-date with Geoengineer.

Sign up for our monthly E-newsletter and you’ll get the latest news on resources, announcements and activities of our Center and affiliates. You can always find our latest edition at our website. You may also unsubscribe from our mailing list at any time.

Promote your firm and its products like never before with Geoengineer.org!

Geoengineer.org offers a wide and unique selection of promotional opportunities. Sponsorship is cost effective, continuous, and international, with benefits ranging from clickable banners/textlinks displayed on the main and other pages of the website and the Newsletter to promotion of news through the Press Center, forums, newsletter, announcement boards, databases and individuals webpages. All GN Members are eligible for sponsoring/advertising at Geoengineer.org at a 10% reduced rate on the annual Sponsorship Programs Fees.

Dimitrios Zekkos, Ph.D., P.E., Managing Director of Geoengineer.org and Assistant Professor, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, USA. Contact e-mail: zekkos@geoengineer.org

GEO INTEREST

the Water in the Soil – Part 3

Bill Hodge

While working on this approach to try and figure out where pore water pressure comes from, and how it might be calculated, the realization that hydrodynamics is a necessary part of Soil Mechanics became increasingly obvious. In hindsight I'm surprised it took me so long to come to appreciate how crucial a partner Fluid Mechanics is of Soil Mechanics, especially when it should have been clear from the start that deformation of a saturated soil-structure is basically a matter of water moving around obstructions.

In the last article I used the Coefficient of Drag [C_D] to calculate the forces acting on a solid as it moved through water. This is of course a term borrowed from our hydraulic colleagues. Now in this article I want to transform that term into our own language and rules of behaviour.

But first, in order to be able to work effectively with hydrodynamics it is necessary to become reacquainted with another of their key terms without which it would be all but impossible to advance.

a Word about the Reynolds Number

I've tried to keep away from this parameter which for most practicing geotechs will be all but a silent echo from student days. But the Reynolds Number [R_E] is too useful to be done without. And there is no good substitute to replace it. Once I found the need to invoke the ideas and tools of hydrodynamics I knew I had to learn to live with the Reynolds Number too.

Back in 1938 Hunter Rouse published a curve relating C_D values for rough spheres to R_E. His curve is reproduced here as the grey line in Figure 7. Fortunately, it covers the full range of practical interest to us.

Conveniently too, as it turns out, for geotechnical purposes the value of R_E is given to an accuracy of two decimal places by the simple multiple:

$$R_E = D\,v$$

provided the diameter "D" is in millimetres, the relative velocity "v" is in millimetres per second, and the water temperature is about 20° C. These conditions result in the combination of the other hydraulic parameters becoming equal to unity.

the Problem with Drag

The idea of simply adopting C_D wholesale made me a bit nervous. Nervous, mainly, because I had no feel for it. It's not something I'd ever used in the field, like cohesion or friction - a parameter I could pull out of my head for a back-of-the-envelope estimate on the run. At liquefaction velocities this parameter can vary from 3,000,000 for fine silt, to less than 0.4 for gravel - and for no good reason intuitively apparent to me.

I felt the need to find some way of relating to the basic physics behind this widely (I might say wildly) varying parameter. And preferably, if it were ever to be confidently adopted in practice, then in a geotechnically analogous way. As it turned out, there are two geotechnical mechanisms with which we are all familiar and which can be used to get quite close to replicating this very useful, but rather intimidating parameter. The two geotechnical analogies I found that fitted the bill were bearing capacity of foundations, and standpipe piezometers.

Bearing Capacity

Anyone who dived into the water from a bit too high up doesn't need to be told that water resists penetration. It's all a matter of speed of entry. This is because water is viscous and therefore its resistance to penetration increases with the rate at which it has to deform. So the thought arose that the interaction of a particle moving in water might be equivalent to steady state bearing capacity displacement, where the strength of the "foundation" was proportional to the rate of straining.

So lets consider that the Drag Force on a sphere, or maybe just some significant fraction of it which I'll call the Bearing component [F_B], is simply equal to the ultimate bearing capacity [q_{ult}] of a circular footing of the same size. In normal terminology this is:

$$F_B = q_{ult}\,A = c\;N_c\;A$$

where: c shear strength

N_c bearing capacity factor

A equatorial area of sphere.

To make this work two shear strength terms need to be related across the disciplines: How could soil shear strength "c" be expressed in terms of water viscosity "μ" ? Both these terms are defined as resistance to shearing force, it is only that the latter is also directly dependent on the rate of straining, and consequently has stress-time

[Pa.s] units. To sort this out requires a bit of mathematical juggling.

Dimensional Analysis

In order to make viscosity dimensionally equivalent to cohesion it needs to be multiplied by one or more parameters, which taken together, have the dimension Time^{-1}. Velocity [m/s] suggests itself as a candidate in this situation, and would, if divided by some significant length "Y" [m], resolve the incompatibility satisfactorily. Therefore, according to dimensional analysis theory, the following equation, where "sv" is some significant velocity, and "Y" is some significant linear dimension, must hold true:

$$c = \mu \; sv / Y$$

$$[Pa] = [Pa.\ s.\ m/s.\ 1/m] = [Pa]$$

Consequently, we may now write:

$$F_B = \mu \; sv \; (N_c / Y) \; A$$

To try to discover what "Y" might be, I did a regression analysis directly comparing this F_B component of Drag Force with the standard equation F_D as given in Part 2. The result was pleasantly surprising. Over a large range of the smaller sphere sizes and lower relative velocities there was complete agreement between the two formulations for Drag Force once "N_c/Y" was given the value "12/diameter" and the velocity "sv" was simply the "v" term representing relative motion between the phases.

So how to make geotechnical sense of N_c/Y = 12/diameter?

We know that for a circular footing N_c has a value between 5.7 and 6.2 when shape factors are included, therefore setting N_c = 6 seems acceptable. And doing that would mean Y = spherical diameter/2, or simply, the radius of the sphere. Therefore, we can now express the equivalent cohesion in the bearing capacity analogy in terms of water viscosity as follows:

$$c = \mu \, v / \text{radius} = 2 \, \mu \, v / D$$

And it follows that the Bearing component of Drag Force becomes:

$$F_B = q_{ult} \; A = c \;\; N_c \; A = (12 \; \mu \; v / D) \; A$$

What this amounts to is that if this component [F_B] were the full equivalent of the Drag Force [F_D] then C_D would equal 24μ/ϱvD. This equivalent value is plotted as the solid red line marked "B" on Figure 7.

Standpipe Piezometer

At any point where flowing groundwater is locally blocked, and made to become stationary, the kinetic energy of the water is transformed into hydraulic pressure. The energy conversion from dynamic to static is given by:

$$h = v^2 / 2 \, g$$

where "h" is the pressure head in metres of water. It represents the additional amount by which the water level in a standpipe piezometer will be increased by being placed within flowing water, as opposed to stationary water. This is because the water level in the standpipe, being static and balanced, must equal the energy level of the flowing water to which it is exposed. The incremental water pressure associated with this condition is obtained by multiplying the head by the unit weight of water [γ_w = ϱ g].

Similarly to what I did above with the Bearing component I now want to consider that the Drag Force, or some fraction of it which I'll call the Pressure component [F_P], is equal to that part of piezometric pressure derived from the velocity head, so that:

$$F_P = (\gamma_w \; h) \; A = (\varrho \, g \, v^2 / 2 \, g) \, A = (\varrho \, v^2 /2) \, A$$

If this component [F_P] were the full equivalent of the Drag Force [F_D] then C_D would equal 1. That is why the blue line marked "P" on Figure 7 is plotted horizontally through unity.

A new term: the L–factor

As noted above, the grey line in Figure 7 is the relationship between C_D and R_E determined by Hunter Rouse back in 1938. What I want to do now is show that this unfamiliar parameter C_D can be replaced with a simple combination of the two geotechnical pressure terms: γ_wh, and q_{ult} .

What we can see/learn from the red line marked "B" on Figure 7 is that setting C_D equal to 24μ/ϱvD, the value from which F_B is derived, this swap provides an exact replication of C_D for any value above about 30 or 40; but thereafter, it is an inaccurate underestimation. This means that replacing the original Drag Force F_D by the Bearing

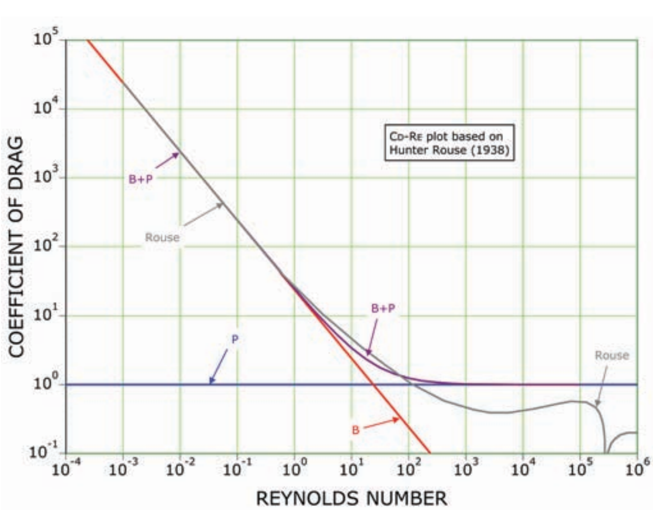


Figure 7. The coefficient of drag.

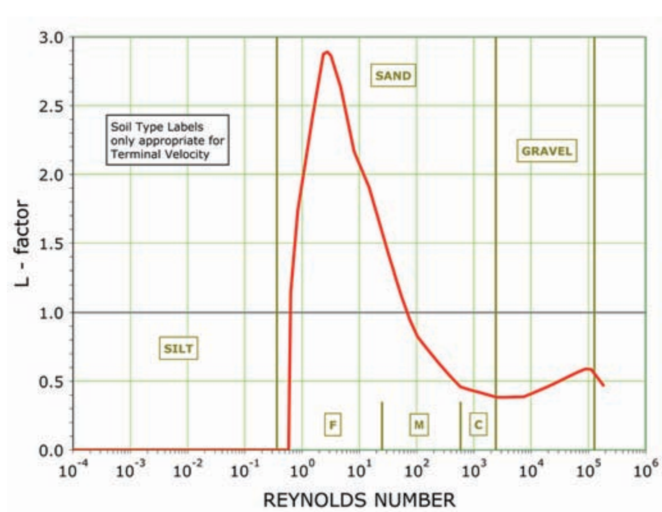


Figure 8. The L-factor.

component F_B is a reasonable thing to do for R_E values up to about 1.

The blue line marked “P” running along the C_D = 1 ordinate in this figure corresponds to assuming F_D could be replaced by the Pressure component equation, F_P, that is assuming $F_D = F_P$. It is obvious that this effort at replication leads to gross underestimations until it cuts the C_D curve at about R_E = 100. Beyond this point it produces equivalent C_D values which are an overestimation by a factor of about 2.

Therefore, the adoption of neither analogy on which these lines are based is acceptable in its own right for the full range of C_D of interest to us.

A purple line (with open circles) “B+P” shows the result of the simple addition of the ordinates of lines B and P: This is equivalent to assuming that the effects of both the Bearing and Pressure components act simultaneously on the particle. By this means the departure from the experimental curve is reduced considerably, to an amount which, in the context of a parameter which has a practical variability of seven orders of magnitude, might be considered a reasonably approximation. Nevertheless, because I want to bring the combined influences of the two components (F_B and F_P) into full alignment with Rouse’s C_D over the full range of R_E it became necessary to introduce and apply a correction factor. This is the “L–factor”.

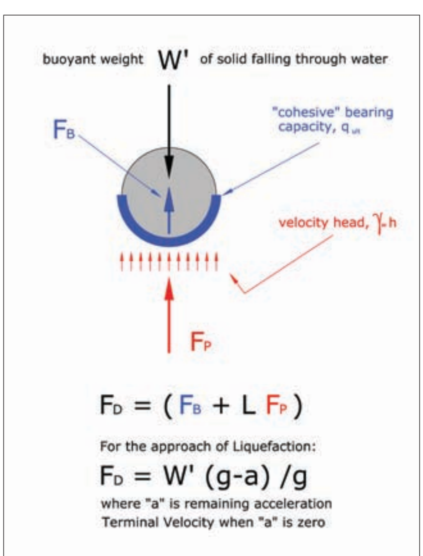


Figure 9. Forces on falling ball.

In applying an alignment factor to the two separate and additive components of Drag there is a choice. Without resulting in any inaccuracy to the value of the Drag Force F_D computed, a non-dimensional L-factor can either be applied as an overall multiplier, as in $L (F_B + F_P)$, or as a component-specific multiplier, as in $(F_B + L F_P)$. At this stage of the development it is more instructive to use the latter alternative, and so, in Figure 8 the appropriate values of the L-factor are plotted for use in the equation:

$$F_D = F_B + L F_P$$

Across the range of interest to us the values of the L-factor vary between 0.0 and 2.9. Here then are the sort of numbers I can keep in my head, something I could never do with C_D, the equivalent values of which vary between 0.39 and 3,350,000 over the same domain.

You will see “soil-type” labels marked across the R_E range in Figure 8. It is necessary to say that these labels apply only at Terminal Velocity. But because up till now we have concerned ourselves mainly with the liquefaction phenomenon I have added them to help put thing into some context.

Modifying Mechanics – From Fluid to Soil

At this stage we can now rewrite the hydraulics style formula for Drag Force, $F_D = C_D \varrho A v^2/2$, in geotechnical terms as follows:

$$F_D = F_B + L F_P$$

where: $F_B = q_{ult} A$

$F_P = \gamma_w h A$

I’ve drawn the free-body diagram in Figure 9 to help illustrate the balance of forces involved in this approach. This shows the formulation for the special case of liquefaction. Later in this series of articles the more general case of soil-structure deformation will be illustrated.

This geotechnical version, which gives exactly the same answers as the original, allows contemplation of solid-to-water interaction in terms of two separate mechanisms with which we are quite familiar ourselves. And now we are free to think of fine particles as gradually settling footings, and to think of gravel as solid impediments confronting the impulse of flowing water.

But there is more to the above than just appropriation of the good work of our hydraulics colleagues. What we might now have at our disposal is a two-part elemental vector pointing along a potential gradient parallel with the thrust of soil-structure distortion. This comes about because the force F_D cited above is generated by each individual particle in that part of the saturated mass which is being moved. It is quite similar to a seepage gradient where water moves through soil under the influence of an external hydraulic gradient. The difference is that in the case of steady state seepage there is no instability or geometric alteration of the soil-structure, whereas what we are dealing with in these articles is pore pressure change brought about by a deforming soil-structures.

In practical terms I find it interesting that for relative velocities around those associated with liquefaction, the L-factor has the following values: across the full silt size range L equals zero; it reaches a peak for fine sands; and then falls to around 0.5 for gravels where turbulent flows are to be expected.

It is a consequence of how the Bearing component was formulated that it may be concluded that the term F_B is not a contributor to pore pressure. Herein, the energy derived from the work done as the Drag Force progresses may be spent entirely in overcoming viscosity, or following the analogy adopted here, “cohesion” and, I suppose, just dissipated as entropy/heat. Similarly, it is consistent to presume that it is only the Pressure component F_P which contributes to pore water pressurization, and this takes place as kinetic energy is converted to static potential on the upstream side of solids confronting the water’s relative velocity. Following this line of reasoning we will proceed from here on the understanding that all to do with excess pore water pressure in soils under deformation is contained in the term F_P, and that F_B is a thing apart.

in the Next Article

Up to this point I've been looking at the behaviour of a single particle falling through water because that, to a large degree, is what I think liquefaction is all about. What I want to do in the next article is to generally conclude my thoughts on this particular type of failure. Also at that time I will suggest: why silts are not as prone to liquefaction as sands seem to be; point out the areas of agreement and conflict between this proposal and the triaxial testing at Harvard and UBC; and, discuss the comparative effects of earthquake shear waves and surface waves on a saturated soil-structure.

Glossary of Terms

$F_D = (C_D\, \varrho\, v^2/2)\, A$

$F_B = (12\, \mu\, v / D)\, A = c\, N_c\, A = q_{ult}\, A$

$F_P = (\varrho\, v^2 / 2)\, A = \gamma_w\, h\, A$

where:

F_D	Drag Force	N
C_D	Coefficient of Drag	-
ϱ	mass density of water	kg/m^3
v	relative velocity	m/s
A	equatorial area of sphere	m^2
F_B	Bearing component	N
μ	viscosity of water	Pa.s
D	diameter of sphere	m
c	cohesion	Pa
N_c	bearing capacity factor	-
q_{ult}	ultimate bearing capacity	Pa
F_P	Pressure Component	N
γ_w	unit weight of water	N/ m^3
h	velocity head	m

W.E. Hodge, Geotechnical Engineer, P.Eng., M.ASCE,
(250) 307 4357, wehodge@shaw.ca

Ram Jack Announces Helical Pile Milestone

Angus W. Stocking, L.S.

Ram Jack Systems Distribution, LLC, based in Ada, Oklahoma, recently became the first (and so far, only) helical pile manufacturer to receive an International Code Council (ICC) Evaluation Service Report (ESR-1854) per the Acceptance Criteria for Helical Foundation Systems (AC358) adopted June 5th, 2007.

Now that ESR-1854 has been approved, engineers and contractors can choose a helical pile that is compliant with the International Building Code and recognized by ICC-ES.

Helical piles were first used in 1836, and by 1900 had been used to support hundreds of marine structures (like lighthouses and ship moorings) in Europe and the United States. Over the last 175 years, the applications for helical piles have been virtually endless. Helical piles are basically a central shaft with one or more helix-shaped bearing plates attached. They're typically installed with hydraulic torque drivers that 'screw' the pile into the ground; extension shafts are added until the desired depth and/or torque are achieved. Integral brackets are then used to attach the pile to structures.

Because helical piles resist both compressive and tension loads, can be installed at any angle, and are effective in problem soils they're one of the most versatile geotechnical tools available to engineers. Helical piles are commonly used to support new and existing residential, commercial and industrial structures, retaining walls, pipelines, boardwalks as well as utility and renewable energy structures. ESR-1854 applies specifically to the Ram Jack 2.875" O.D. x 0.217" wall thickness pile. "It's our most popular product, and hundreds of thousands have been installed in the Americas," says Willis, "But of course we're working on updating our ESR to include more of our product line, starting with larger diameters."

Before AC358 was adopted by ICC-ES, there was not a common standard for helical pile systems in the industry. Most manufacturers only provided capacities of individual components and not the capacity of how the system worked as a unit. A handful of manufacturers, including Ram Jack, had obtained certification under various agencies and these manufacturers were granted 'legacy' status after the ICC was formed in 2000. But this status didn't provide a standard guideline. In 2005, some helical pile manufacturers began to work with the ICC-ES and private consultants and that work led to official acceptance criteria.

ESR-1854 also applies to Ram Jack's 2.875" push pile, a move that surprised some industry observers. "It makes sense," says Willis, "Our push piles have the same shaft and bracket configuration as the helical pile, minus the bearing plates, and it's only those parts of the push pile that are certified. They only work in compression but are ideal for remedial repair of existing structures." The ESR extension to push piles gives engineers even more versatility when designing appropriate foundation systems.

Ram Jack has been designing and making steel piles for more than 40 years, but was primarily a contracting firm until the 1980s.

Angus W. Stocking, L.S., 207A S. University Drive, Beaver Dam WI 53916, 270.363.0033 cell 920.219.9544 home/office, angusstocking@gmail.com., www.InfrastructureWriting.com

Angus W. Stocking, L.S. is a licensed land surveyor now writing fulltime on infrastructure topics.

THESIS
ABSTRACTS

Comparison of Geoenvironmental Properties of Caustic and Noncaustic Oil Sand Fine Tailings

Warren Gregory Miller

Warren Gregory Miller, Worley Parsons, Warren.miller@worleyparsons.com

A study was conducted to evaluate the properties and processes influencing the rate and magnitude of volume decrease and strength gain for oil sand fine tailings resulting from a change in bitumen extraction process (caustic versus non-caustic) and the effect of adding a coagulant to caustic fine tailings.

Laboratory flume deposition tests were carried out with the objective to hydraulically deposit oil sand tailings and compare the effects of extraction processes on the nature of beach deposits in terms of geometry, particle size distribution, and density. A good correlation exists between flume deposition tests results using oil sand tailings and the various other tailings materials. These comparisons show the reliability and effectiveness of flume deposition tests in terms of establishing general relationships and can serve as a guide to predict beach slopes.

Fine tailings were collected from the various flume tests and a comprehensive description of physical and chemical characteristics of the different fine tailings was carried out. The characteristics of the fine tailings is presented in terms of index properties, mineralogy, specific surface area, water chemistry, liquid limits, particle size distribution and structure. The influence of these fundamental properties on the compressibility, hydraulic conductivity and shear strength properties of the fine tailings was assessed. Fourteen two meter and one meter high standpipe tests were instrumented to monitor the rate and magnitude of self-weight consolidation of the different fine tailings materials. Consolidation tests using slurry consolidometers were carried out to determine consolidation properties, namely compressibility and hydraulic conductivity, as well as the effect of adding a coagulant (calcium sulphate [CaSO4]) to caustic fine tailings. The thixotropic strength of the fine tailings was examined by measuring shear strength over time using a vane shear apparatus.

A difference in water chemistry during bitumen extraction was concluded to be the cause of substantial differences in particle size distributions and degree of dispersion of the comparable caustic and non-caustic fine tailings. The degree of dispersion was consistent with predictions for dispersed clays established by the sodium adsorption ratio (SAR) values for these materials. The biggest advantage of non-caustic fine tailings and treating caustic fine tailings with coagulant is an increased initial settlement rate and slightly increased hydraulic conductivity at higher void ratios. Thereafter, compressibility and hydraulic conductivity are governed by effective stress. The chemical characteristics of fine tailings (water chemistry, degree of dispersion) do not have a significant impact on their compressibility behaviour and have only a small influence at high void ratio (low effective stress). Fine tailings from a caustic based extraction process had relatively higher shear strengths than comparable non-caustic fine tailings at equivalent void ratios. However, shear strength differences were small and the overall impact on consolidation behaviour, which also depends on compressibility and hydraulic conductivity, is not expected to be significant.

Supervisors: D. Sego, J.D. Scott, Dept. of Civil & Environmental Engineering, University of Alberta

Subsurface Behavior of Spilled Fuel in a Permafrost Environment

Olumide Iwakun

Olumide Iwakun, oiwakun@civilsol.com

This dissertation focuses on the subsurface behavior of spilled fuel consisting of diesel and gasoline, which are subsets of light nonaqueous phase liquids (LNAPLs), in a permafrost environment. Particular emphasis is laid on mobile LNAPL in fractured bedrock. The site chosen for this study is the abandoned Colomac gold mine, 220 kilometers northwest of Yellowknife in the Northwest Territories, where over 50,000 liters of spilled fuel occurred between 1990 and 2003. The site is underlain by fractured bedrock with 0 to 4.6 m of overburden soil. The broad objectives of this work involve determination of contamination extent and LNAPL behavior at the site. Other specific objectives include determination of the major geochemical processes and identification of mechanisms influencing LNAPL movement and accumulation at the site.

Both field and laboratory studies were performed to achieve the above-stated objectives. The field study involved site characterization, and the laboratory study involved a topdown freezing experiment using a freezing cell, consisting of parallel glass plates, to evaluate the impact of cyclic freeze-thaw on LNAPL movement. The site characterization efforts showed that the LNAPL contamination is limited to the upper section (~7 m) of the fractured bedrock. The field study showed that water table fluctuations and freezing-induced displacements were active but discontinuous mechanisms contributing to LNAPL migration and accumulation in the formation and monitoring wells at the site.

Analyses of the groundwater suggested ongoing anaerobic biodegradation of the dissolved LNAPL components. Furthermore, the analyses showed that the water was CaSO4 type and the main geochemical processes were gypsum dissolution and carbonate weathering. The analyses underscored the importance of bedrock mineralogical composition on groundwater constituents and geochemical processes.

The laboratory test results involving entrapment of diesel fuel below the water column and admixture of soluble oils with water in the freezing cell showed upward mobility of LNAPL under cyclic freezing, and downward progressive expulsion of the soluble oils ahead of the advancing freezing front. The results corroborated literature findings on cryogenic expulsion ahead of freezing front, and

provided new insight into the behavior of trapped LNAPL below the water table when subjected to cyclic freezing.

Supervisor: D. Sego, Dept. of Civil & Environmental Engineering, University of Alberta

Earth Pressures and Loads on Induced Trench Culverts

Benjamin L McGuigan

Benjamin L McGuigan, GEMTEC Limited, 191 Doak Road, Fredericton, New Brunswick E3C 2E6, Tel: 506-453-1025, Fax: 506-453-9470, email: benjamin.mcguigan@gemtec.ca

Centrifuge tests were performed to measure earth pressures on a single box culvert installed in several induced trench configurations. A parametric study performed with numerical modelling identified a preferred compressible zone geometry having a width of 1.2 times the culvert width (Bc) and a thickness of half the culvert height. The earth pressure on top was 0.28 times the overburden. The induced trench base contact pressures were 50% higher than the top pressures plus dead load due to drag forces mobilized along the sidewalls; however, they were 35% less than the base contact pressures for the positive projecting condition.

A numerical model calibrated with centrifuge test results was used to evaluate the effects of culvert spacing and compressible zone geometry on twin induced trench box culvert pressures. One compressible zone spanning both culverts was preferred for culverts spaced at 0.5-1.0Bc, while two zones, 1.2Bc wide, were preferred for 1.5Bc spacing. The base contact pressures were 41-47% lower than for the corresponding positive projecting cases.

A field instrumentation of twin 3660-mm diameter culverts installed in an induced trench under 21.7 m of fill was also performed. Earth pressures were measured at the crown, shoulder, springline, haunch, and invert locations. Average crown and springline pressures were 0.67 and 0.35 times the overburden, respectively. Poor compaction in the haunch regions and stiff bedding conditions led to stress concentrations at the inverts. Numerical modelling was used to determine a vertical earth load of 0.87 times the soil prism load, which was 30% lower than for the positive projecting condition.

Induced trench construction therefore appears to be viable for single and twin box culverts, provided that drag forces along the sidewalls are accounted for in the design, as well as for large diameter twin pipes, provided that bedding and constructability issues are addressed.

Supervisor: Dr. Arun J. Valsangkar, Professor Emeritius, Dept. of Civil Engineering, University of New Brunswick

Effect of Reinforcement and Soil Viscosity on the Behaviour of Embankments over Soft Soil

Chalermpol Taechakumthorn

Chalermpol Taechakumthorn (c.taechakumthorn@gmail.com)

A verified elasto-viscoplastic finite element model is used to develop a better understanding of the performance of geosynthetic reinforced embankments over rate-sensitive soil. For rate-sensitive soils, the generation of creep-induced pore pressures following the end of construction is evident along the potential slip surface. As a result, the minimum safety factor with respect to embankment stability occurs after the end of construction. The combined use of reinforcement and PVDs are shown to provide an effective means of minimizing creep-induced excess pore pressure, increasing overall stability, and decreasing deformation of the embankments.

The combined effects of the viscoelastic properties of geosynthetic reinforcement and the rate-sensitive nature of foundation soils on the performance of embankments are examined. The effect of various factors, including reinforcement type, soil viscosity, construction rate and allowable long-term reinforcement strain, on the time-dependent behaviour of reinforced embankments are considered. From a series of finite element analyses, the ideal allowable reinforcement strains to minimize embankment deformation while providing optimum long-term service height of the embankment, considering the effect of soil and reinforcement viscosity, are proposed for soils similar to those examined in this study.

The currently proposed design methods for embankments with creep-susceptible reinforcement constructed over rate-sensitive soils appears to be overly conservative. This study proposes a refined approach for establishing the allowable long-term reinforcement strains that are expected to provide adequate performance while reducing the level of conservativeness of reinforced embankment design.

Finally, a previously developed elasto-viscoplastic constitutive model is modified to incorporate the effect of soil structure using a state-dependent fluidity parameter and damage law. The model was evaluated against data from a well-documented case study of a reinforced test embankment constructed on a sensitive Champlain clay deposit in Saint Alban, Quebec. The benefit of basal reinforcement and the effect of reinforcement viscosity are then discussed for these types of soil deposits.

Advisor: Dr. R. Kerry Rowe, Professor and Canada Research Chair in Geotechnical and Geoenvironmental Engineering, Queen's University

Diffusive Transport of Volatile Organic Compounds through Geomembranes

Rebecca S. McWatters

Rebecca S. McWatters, Postdoctoral Fellow, GeoEngineering Centre at Queen's-RMC, Queen's University, Kingston, ON, Canada, and Monash University, Melbourne, VIC, Australia, and the Australian Antarctic Division, Kingston, TAS, Australia, email: rebecca.mcwatters@aad.gov.au

The diffusive transport of volatile organic compounds (VOCs) through geomembranes is examined. The key diffusive parameters: diffusion (Dg), partitioning (Sgf) and permeation (Pg) coefficients, for transport from both vapour and aqueous phases are evaluated. Consideration is given to different types of geomembrane, exposure to cold climatic conditions, and aged geomembranes exhumed after 3 and 25 years. Laboratory sorption and diffusion tests are performed and modeling is used to infer diffusive parameters from experimental data. The transport of VOCs through polyvinyl chloride (PVC) and linear low-density polyethylene (LLDPE) geomembranes from both aqueous and vapour phases is evaluated by Purge & Trap-GC/MS. Results indicate that VOC transport through geomembranes in a simulated landfill environment is identical despite the phase they originate from. Subsequently, this finding is confirmed by examining diffusion of vapour-phase VOCs using Solid Phase Microextraction-GC/FID.

Diffusive transport of VOCs through traditional PVC, LLDPE and high-density polyethylene (HDPE) geomembranes is compared with that through two novel co-extruded geomembranes, one with a polyamide inner core, the other an ethylene vinyl alcohol (EVOH) inner core. Both co-extruded geomembranes show a 10-200-fold decrease in Pg values and therefore improved diffusive resistance to VOCs compared to the traditional geomembranes. EVOH also shows a 5-12-fold decrease in Pg values compared to an HDPE geomembrane.

The effects of cold environments on the diffusion of VOCs are studied. Five geomembranes are exposed to simulated cold climatic conditions in the laboratory. Results from diffusion tests run at 2-24oC indicate Dg and Pg decrease with temperature. The temperature and diffusion coefficients relationship follow the Arrhenius equation. Activation energies of diffusionare calculated specific to each geomembrane and contaminant. An HPDE geomembrane taken from a field site in the Canadian Arctic after three years of exposure to old climatic conditions shows minimal decreases in Dg and Pg when compared to new HDPE. Finally, the diffusion of VOCs through an HDPE geomembrane exhumed from a decommissioned PCB landfill originally built in 1984 is examined. Profiling of PCB concentrations in the landfill clay and composite liners is investigated indicating minimal PCB diffusive migration after 25 years.

Advisor: Dr. R. Kerry Rowe, Professor and Canada Research Chair in Geotechnical and Geoenvironmental Engineering, Queen's University

Three-Dimensional Kinematic Controls on Rock Slope Stability Conditions

Marc-André Brideau

Marc-André Brideau, mbrideau@auckland.ac.nz, +64 027 244 7867, School of Environment, University of Auckland, Private Bag 92019, Auckland 1142, New Zealand

This thesis investigates the three-dimensional influence of discontinuity sets and topography on kinematics of rock slope stability and failure mechanisms. A field data collection methodology was developed to provide the inputs to a slope stability investigation that utilises three-dimensional geometric, limit equilibrium and distinct element codes. Conceptual slope geometries in addition to three case studies are employed to evaluate the influence of discontinuity set orientation and lateral kinematic confinement on the failure mechanism and slope stability conditions. The influence of varying the discontinuity persistence and block size in a three-dimensional distinct element code are also investigated. Systematic studies of these parameters are performed for the planar sliding and block toppling failure mechanisms. This thesis presents the first detailed description and slope stability analysis of the McAuley Creek Landslide and the Chehalis Lake Landslide. New data and analyses of the potentially unstable rock mass at Third Peak on Turtle Mountain are also presented. Two recently developed representations of complex topography in the three-dimensional distinct element code are applied to the case studies. The results obtained in this thesis are compared to the description of other local and international large rock slope failures published in the literature.

Keywords: slope stability; failure mechanism; numerical modelling; three-dimensional; limit equilibrium; block theory; distinct element code.

Supervisor: Professor Doug Stead, Department of Earth Sciences, Simon Fraser University

Multi-Scale Characterization of Rock Mass Discontinuities and Rock Slope Geometry Using Terrestrial Remote Sensing Techniques

Matthieu Sturzenegger

Matthieu Sturzenegger, Klohn Crippen Berger, 500 – 2955 Virtual Way, Vancouver, British Columbia, V5M 4X6, email: msturzen@alumni.sfu.ca

Terrestrial remote sensing techniques including both digital photogrammetry and laser scanning, represent useful complements to conventional field mapping and rock mass discontinuity characterization. Several studies have highlighted practical advantages at close-range (< 300 m), including the ability to map inaccessible rock exposures and hazard reduction related to both traffic and rockfall along investigated outcrops. In addition, several authors have demonstrated their potential to provide adequate quantification of discontinuity parameters. Consequently, their incorporation into rock slope stability investigations and design projects has grown substantially over recent years.

As these techniques are increasingly applied by geologists and geological engineers, it is important that their use be properly evaluated. Furthermore, guidelines to optimize their application are required in a similar manner to standardization of conventional discontinuity mapping techniques. An important thesis objective is to develop recommendations for optimal applications of terrestrial remote sensing techniques for discontinuity characterization, based on a quantitative evaluation of various registration approaches, sampling bias and extended manual mapping of 3D digital models.

It is shown that simple registration networks can provide adequate measurement of discontinuity geometry for engineering purposes. The bias associated with remote sensing mapping is described. The advantages of these techniques over conventional mapping are demonstrated, including reliable discontinuity orientation measurements. Persistence can be precisely quantified instead of approximately estimated, resulting in a new class for extremely persistent discontinuities being suggested. Secondary roughness and curvature can also be considered at larger scales. The techniques are suitable for the definition of discontinuity sets, and the estimation of both trace intensity and block size/shape, if sampling bias is correctly accounted for. A new type of sampling window, suitable for the incorporation of remote sensing data into discrete fracture network models is presented.

Another significant thesis objective is the extension of terrestrial digital photogrammetric methods to greater distances (> 1 km), using f = 200-400 mm lenses. This has required a careful investigation of the observation scale effects on discontinuity parameters. The method has been applied in a large open-pit mine and on the Palliser Rockslide. It allows detailed characterization of the failure surfaces, volume estimations and pre-slide topography reconstruction.

Keywords: rock mass discontinuity characterization; terrestrial digital photogrammetry; terrestrial laser scanning; sampling bias; observation scale; Palliser Rockslide; Frank Slide; Palabora Open Pit Mine; failure surface; registration

Supervisor: Professor Doug Stead, Department of Earth Sciences, Simon Fraser University

Exploring the Influence of Tactical Airfield Layer Properties on Light Weight Deflectometer Response

Christopher T. Senseney

Christopher T. Senseney, csensene@mines.edu

The United States Air Force has traditionally used dynamic cone penetrometer (DCP) evaluation of tactical airfields coupled with empirical design methods. There are limitations with the DCP, e.g., penetration through cobbles, noisy, and time consuming. Additionally, the pavement community is moving towards mechanistic-empirical design methods wherein layer moduli are the primary input parameter. The light weight deflectometer (LWD) is a device that can assess soil moduli without the need to penetrate the soil surface, and to perform testing in a quieter and timelier fashion. The interpretation of LWD data to characterize layer parameters is in its infancy. This thesis aims to advance the understanding of layered soil response to LWD loading and to develop an improved methodology to obtain layer parameters from the LWD test.

The conventional LWD test includes a single sensor to measure vertical deflection at the plate center wherein a static analysis is performed using peak deflection and force to estimate a representative deformation modulus. To improve the conventional LWD test, the first topic of this thesis evaluates the use of radial offset sensors to backcalculate layer moduli of a two-layer soil system using a static analysis. The measurement depth for the LWD with radial offset sensors was 1.8 times plate diameter versus the conventional measurement depth of 1.0 to 1.5 times plate diameter. The second topic of this thesis presents a dynamic finite element (FE) model of the LWD test. Inertia and energy dissipation of the soil are neglected in a static analysis, and it was found that results from a static analysis can be substantially different than results from a dynamic analysis. By performing a sensitivity analysis of the FE model, it was found that damping ratio and Poisson's ratio have similar influence on peak deflections; however, their influence is an order of magnitude less than that of elastic modulus. The third topic of this thesis presents a genetic algorithm (GA) to backcalculate layer parameters without knowing top layer thickness a priori. The backcalculated parameters from the GA agree with values that were expected, but computational time of the GA was prohibitively long for field application.

Sponsor: Dr. Michael Mooney, Colorado School of Mines

Dynamic Properties of Colloidal Silica Soils Using Centrifuge Model Tests and a Full-Scale Field Test

Carolyn T. Conlee

Carolyn T. Conlee, 41 Pickle Brook Road, Bernarsville, NJ 07924, Tel: 908-642-8731, email: ctc5681@gmail.com

Traditional ground improvement methods to mitigate the effects associated with liquefaction damage are often not feasible in developed areas. Commonly used soil improvement methods can have adverse affects on the surrounding infrastructure and less invasive methods are therefore required. Passive site stabilization is a non-invasive grouting technique where a stabilizing material can be injected at the edge of a site and delivered to target locations through the groundwater. As the stabilizer flows through the subsurface, it displaces the pore water and subsequently forms a permanent gel that binds to soil particles, resulting in a stronger soil formation.

Based on its unique characteristics, colloidal silica has been selected as an ideal material for passive site stabilization. For purposes of liquefaction mitigation, the dynamic behavior of colloidal silica soils was studied through centrifuge model tests and a complementary, full-scale field test. The centrifuge tests provided comparisons of the response for untreated sands and sands treated with 4%, 5%, and 9% colloidal silica concentrations subjected to a sequence of dynamic shaking events. To complement the model tests, a full-scale field test was conducted to compare the response of a liquefiable soil formation to a soil grouted with colloidal silica. Permeation grouting techniques and field procedures were developed in order to treat an approximately 1.5 m (5 ft) thick liquefiable soil layer.

The centrifuge model tests and field test both show that colloidal silica soils reduce settlement, lateral spreading, and shear strains induced when subjected to large dynamic loads. For purposes of developing soil models, shear modulus degradation curves were developed and relationships that govern unloading-reloading behavior were identified in centrifuge model tests. Amplification in the acceleration response and increases in excess pore pressure ratios were determined to be direct indications of treatment levels. Large transient changes observed in pore pressure response were shown to describe the behavior of stress transmittal between the soil and gel during cyclic loading. Additionally, the hysteretic response of colloidal silica soils exhibited greater hysteretic damping and cyclic mobility consistent with dense sands. The response also revealed a lower degree of cyclic degradation for higher concentrations of colloidal silica.

Advisor: Patricia M. Gallagher, Dept. of Civil & Environmental Engineering, Drexel University

Investigation of Sediment Erosion Rates of Rock, Sand, and Clay Mixtures Using Enhanced Erosion Rate Testing Instruments

Raphael Crowley

Raphael Crowley, Post-Doctoral Associate; University of Florida, Department of Civil & Coastal Engineering, 365 Weil Hall, Gainesville, FL 32611; Tel: 352-392-3261; Fax: 352-392-9537; email: rcrowley@ufl.edu

Scour is the primary cause of bridge failures in the United States. Although predicting scour depths for non-cohesive (sandy) bed materials is fairly well understood, much less is known about predicting scour depths when cohesive materials such as clays, sand-clay mixtures, and rock are present. A semi-empirical method exists for predicting cohesive scour depths. This method relies on the input of a sediment transport function or erosion rate as a function of shear stress. Current design guidelines such as HEC-18 recommend measuring sediment transport functions in a laboratory, but there has been some question as to how to do this properly.

To answer this question, a series of improvements and enhancements were made to the Sediment Erosion Rate Flume (SERF) at the University of Florida (UF). A laser leveling system, a vortex generator, a shear stress measuring system, computer updates, and a sediment control system were designed and implemented. Using the new shear stress system, a series of tests were run to assess the proper way to measure shear stress in a flume-style erosion rate testing device. Results showed that the pressure drop method will not measure shear stress properly, and in the absence of a shear stress sensor, the most effective alternative method for estimating shear stress is to use the Colebrook Equation (which describes the Moody Diagram). A new material was developed for testing in both the SERF and the Rotating Erosion Testing Apparatus (RETA) to serve as a basis of comparison between the two instruments. Results were inconclusive because rock-like erosion described by the Stream Power Model appeared to dominate erosion behavior. A database of results from the RETA that has been developed since the RETA's inception in 2002 was used to verify that it is measuring the correct erosion rate vs. shear stress relationships. Results showed that for the special case where particle-like erosion dominates, the RETA appears to produce correct results. Results also appear to indicate that when rock-like erosion is present, it is generally an order of magnitude lower than situations where particle-like erosion dominates. Further analysis of the database showed that there may be a correlation between material strength and erosion rate. Further research was aimed at generalizing erosion rate vs. shear stress relationships for sand-clay mixtures. A series of tests were conducted on a variety of sand-clay mixtures. Results showed sensitivity to the method in which the sand-clay mixtures were prepared. Rock-like erosion and particle-like erosion were present in most sand-clay mixtures even though typical sand-clay mixtures would not typically be described as "rock-like materials." Recirculating sediment during sand-clay testing indicated that suspended sediment in the SERF has little effect on bed shear stress.

Sponsor: Dr. David Bloomquist; Associate Professor, Department of Civil & Coastal Engineering, University of Florida

Carbonate Diagenesis and Chemical Weathering in The Southeastern United States: Implications on Geotechnical Behavior

Joan M. Larrahondo

Joan M. Larrahondo, email: joan.larrahondo@gmail.com

The Savannah River Site (SRS) soils below 30m are marine-skeletal, calcium carbonate-rich, Eocene sediments with varying clastic content and extensive diagenetic alteration. In addition, the SRS' coarse-grained surface soils are rich in chemical weathering-derived iron oxides with various degrees of maturity. This research is aimed to investigate the nature and origin of the diagenetically-altered soft and hard geomaterials that coexist in the SRS area. Another objective is to use laboratory-prepared soil analogs to study the effect of iron oxide coatings on coarse-grained soil mechanics. Geologic surveys encountered meter-sized vugs closely associated with hard limestone. EDS, XRD, SEM, K-Ar dating, P-wave velocities, tensile strength studies, and solubility experiments with ICP-OES analysis highlighted the contrast between hard and soft facies and allowed calculation of kinetic parameters for modeling. The SRS' iron oxide-rich surface soils were laboratory-simulated using Ottawa sands that were chemically coated with goethite and hematite. Small-strain stiffness (Bender Element Technique), large-strain strength, and surface properties (SEM and AFM) were studied on the resulting soil analog. Results indicate that limestone and soft carbonate soil bear distinct geochemical signatures: unlike the soft soil, the limestone exhibits higher crystallinity, lower clastic load, and freshwater-influenced composition. Iron-oxide coated sands deliver distinct inherent fabric parameters when compared to those of uncoated sands. Likewise, small-strain stiffness and critical state parameters are enhanced upon iron-oxide coating. These results reveal a carbonate diagenesis path driven by geologic-time seawater/freshwater cycles, though incompletely justified by inorganic processes alone. Thus, microorganism-driven micritization (or the lack thereof) in early, shallow-marine environment, followed by freshwater micrite lithification are also proposed. Contact mechanics analyses suggest that iron oxide coatings yield an increased number of grain-to-grain contacts, higher surface roughness, and interlocking, which are responsible for the enhanced stiffness and strength properties observed. A behavioral hierarchy associated with iron oxide thermodynamic stability may also exist.

Advisor: Dr. Susan E. Burns, Georgia Institute of Technology Sponsor: US Department of Energy

Gas Production from Hydrate-Bearing Sediments: Geo-Mechanical Implications

Jong-Won Jung

Jong-Won Jung, Ph.D., Earth Sciences Division, MS 70108B, Lawrence Berkeley National Laboratory, 1 Cyclotron Rd. Berkeley CA 94720. email: jwjung7695@gmail.com or jjung@lbl.gov, Office: 1-510-486-4566, Fax: 1-510-486-7152

Gas hydrate consists of guest gas molecules encaged in water molecules. The most common great molecule is methane. Methane hydrate reserves around the world are estimated in 20,000 trillion m3 of CH4. Methane hydrate can be an energy resource, as well as a cause for global warming and the seafloor instability. Gas hydrates occur in sediments under high fluid pressure (~ 25MPa) and low temperature (0 to ~25°C). The properties of hydrate bearing sediments are poorly understood. In particular, gas production from hydrate bearing sediments can cause problems related to over pressure, clogging, fine migration, volume change, and seafloor instability. Research documented in this thesis explores the formation and growth of gas hydrates at the pore scale including wetting condition, formation rate, particle-scale hydrate-mineral tensile strength, and its impact on sediment-scale properties, and volume change during hydrate formation and dissociation. Experimental studies, numerical simulations using PFC-3D, and macro-scale analysis of coupled processing are complemented with particle-scale. Emphasis is placed on identifying the advantages/disadvantages of different gas production strategies such as depressurization, heating, and chemical injection; the later is focus of a fundamental study to enhance the understanding of CH4-CO2 exchange as a unique solution to recovery CH4 gas recovery while sequestering CO2.

Sponsor: J. Carlos Santamarina, School of Civil and Environmental Engineering, Georgia Institute of Tehcnology

Micromechanical Analysis of Geosynthetic-Soil Interaction Under Cyclic Loading

Anil Bhandari

Anil Bhandari, Project Manager, Terracon Consultants, Inc. Charleston, SC, email: abhandari@terracon.com

Geosynthetics have been used to improve the performance of civil engineering instrastructure in many projects. The interaction between geosynthetics and soil is an important factor that governs the performance of the geosynthetic-reinforced structures. Previous studies on geosynthetic-soil interaction using laboratory and continuum based numerical approach were beneficial for studying the overall behavior of the system, however those investigations did not provide insight into microscale response. To improve the understanding of the geosynthetic reinforcement mechanisms, geosynthetic-soil interaction was studied under a monotonic and a cyclic loading using a micromechanical approach.

The micromechanical parameters of the granular materials and reinforcements were calibrated using a biaxial test and a tensile test, respectively. The behavior of granular materials was evaluated under a monotonic and a cyclic loading and analyzed from force and fabric orientation perspectives. Using the calibrated micromechanical parameters, benchmark trapdoor experiments were simulated to establish the simulation techniques for geosynthetic-soil interaction. The micromechanical studies of three practical problems involving geosynthetic-soil interaction were conducted. The practical problems were: geosynthetic-reinforced embankments overlying voids, Geosynthetic-Reinforced Pile-Supported (GRPS) embankments, and geosynthetic-reinforced bases.

In the trapdoor experiments, soil arching was observed as an essentially meta-stable condition. The inclusion of reinforcement in the embankments reduced the settlements measured on the top of the embankments. Geosynthetic reinforcement increased the load transfer to the piles and reduced the load on the compressible soils. The anchorage failure of the reinforcement also controlled the load transfer particularly in the low embankments. In the geosynthetic-reinforced base simulation, the density of base course had a profound effect on a rut depth. The tensile stresses developed in the geosynthetic reinforcement helped distribute the contact forces wider. A relatively small tensile stress developed in the reinforcement; therefore, a very stiff reinforcement was not necessary to improve the performance of the base. An optimum ratio between the aperture sizes to the aggregate diameter was identified for the improved performance of the geogrid-reinforced base.

Sponsor: Prof. Jie Han, The University of Kansas

Analysis and Behavior of Preexisting Landslides

Manzoor Hussain

Manzoor Hussain, Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, 205 N. Mathews, Urbana, IL 61801, Tel: 217-953-4303, email: mhussain@mce.nust.edu.pk

The selection of shear strength parameters for the design and repair of landslide is important and difficult. Skempton (1964) concludes that if a failure has already occurred in clayey soils, any subsequent movement along the preexisting slip surface will be controlled by the drained residual strength. Skempton (1985) suggests that the strength of a clay also will be at or close to the residual value on slip surfaces in soliflucted slopes, bedding shears in folded strata, sheared joints or faults, and after an embankment failure. Therefore, the drained residual shear strength has been and still is being used for analysis of slopes that contain a preexisting shear surface.

The main objectives of this research are to study the shear strength and long term behavior of landslides and in particular preexisting shear surfaces. The research involves laboratory testing to determine the strength recovery, if any, of cohesive soils with varying plasticity and the applicability of the recovered strength to remedial measures and the back-analysis of landslides. Some of the issues addressed include (a) if the shear strength increases from the residual value with time, (b) how long does it take to reach the maximum recovered strength, (c) if the strength increases with time, does the strength return to the residual value with additional shear displacement and if so how much shear displacement is required to reduce the strength back to the residual value, and (d) what is the maximum shear strength that can be obtained from strength recovery and used for design purposes.

Back-analysis of landslides is important for evaluating the mobilized recovered strength and thus back-analysis procedures were reviewed and augmented. Empirical correlations for drained residual and fully softened friction angles proposed by Stark et al. (2005) in graphical form are being widely used in geotechnical practice. These empirical correlations were augmented with additional test data and mathematical equations were developed. These mathematical expressions were incorporated in a spreadsheet that can be used as a tool to estimate the shear strength parameters of a soil by using only two index properties, i.e., liquid limit and clay-size fraction. Thus, the values of shear and effective normal stresses obtained from the spreadsheet can also be used to model the residual failure envelope in a stability analysis.

Sponsor: Timothy D. Stark, University of Illinois at Urbana-Champaign

Three Dimensional Slope Stability Analyses for Natural and Manmade Slopes

Kamran Akhtar

Kamran Akhtar, Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, 205 N. Mathews Ave., Urbana, IL 61801, Tel: 217 953-4303, email: kakhtar2@illinois.edu

This study focused on the importance of three dimensional (3D) slope stability analyses in practice. The results show that if the actual shear strength is used in the design of a slope, a 2-D analysis will yield conservative results. However, a 2D analysis may lead to an overestimate or unconservative value of back-calculated shear strength. 3D analyses are important in back-analyses to accurately assess the relative effects of slope changes, precipitation, shear resistance and remedial measures.

Present 3D limit equilibrium (LE) software does not consider the effects of shear resistance offered by vertical sides that parallel the direction of movement of a translational landslide mass. Based on results of a parametric study conducted herein using finite element (FE) and finite difference (FD) analyses, it was found that the use of an earth pressure coefficient ($K\tau$) that is in-between at-rest (KO) and active (KA) earth pressure provides a reasonable estimate of the side shear resistance and 3D/2D FS ratios that are in agreement with FE and FD analyses. Charts developed herein can be used to determine the importance of performing a 3D slope stability analysis for a translational failure. Failure surfaces for rotational landslides usually do not require the effects of side shear resistance to be included because shear resistance is calculated along the non-vertical sides of the failure surface.

A LE methodology for calculating the 3D factor of safety for natural and manmade slopes and an accompanying user friendly software package were developed. 3DDEM- Slope was developed as part of this study to incorporate and/or verify some of the findings of this study. A comparison of different 2D and 3D slope stability methods e.g., LE, FE, and FD methods, was used to verify the methodology. Using case histories, 2D and 3D slope stability analysis were performed by LE, FE, and FD methods to investigate the applicability and/or limitations of each method to different field slope stability problems and geometries.

Sponsor: Timothy D. Stark, University of Illinois at Urbana-Champaign

Strength of Transversely Isotropic Rocks

Jianyong Pei

Jianyong Pei, Baker Hughes, 2001 Rankin Road, Houston, TX 77073, Tel: 713-966-3811, email: jianyong.pei@bakerhughes.com

This thesis proposes a new Anisotropic Matsuoka-Nakai (AMN) criterion to characterize the failure of transversely isotropic rocks under true triaxial stress states. One major obstacle in formulating an anisotropic criterion is that it usually involves six stress components, instead of three principal stresses. As such, anisotropic criteria usually lead to complicated mathematical expressions, and cannot be directly visualized in three-dimensional space. This problem is solved by introducing the Material Normal Stress System (MNSS), which is the space formed by the three normal stress components reflecting the material anisotropy. Within this system, the failure behavior of transversely isotropic rocks in conventional triaxial tests can be represented with geometrical features in the MNSS. These features are then incorporated into the failure surface of the original Matsuoka-Nakai criterion in the Material Normal Stress System, resulting in the Anisotropic Matsuoka-Nakai criterion. This criterion, combined with the Coulomb criterion, is validated against both conventional and true triaxial test data, that are collected from an extensive literature review. The combination of the AMN criterion and the Coulomb criterion satisfactorily characterizes the measured strength from an extensive program of true triaxial tests on a schist, which confirms the ability of the proposed criterion. Finally, this combination of criteria is applied to the borehole stability problem. The necessary mud pressure against borehole collapse and the onset of borehole failure are examined.

Sponsors: Prof. Herbert H. Einstein, Prof. Andrew Whittle, MIT

The Small Strain Characterization of Unbound Highway Base Course Materials

Torsten Mayrberger

Torsten Mayrberger, 8165 Kronos Dr., Anchorage, AK 99502, email: tmayrberger@pndengineers.com, Tel: 907.887.9401

This dissertation, The Small Strain Characterization of Unbound Highway Base Course Materials, investigated the influence of several variables on the resilient modulus of unbound base course aggregate materials. The following variables that affect the resilient modulus were investigated: gradation, varying degrees of saturation and undrained Conditions, parent material type, stress ratio and stress history, resilient Poisson's ratio and volume change, and the effects of data acquisition methods on final results.

This research program utilized testing protocols AASHTO T 307 The Standard Method of Test for Resilient Modulus of Unbound Granular Base/Sub-base Materials and Sub-grade Soils (AASHTO 2002) and NCHRP Project 1-28A Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design - Task II: Unbound Materials (NCHRP 2004). Unique test protocols and methods were also developed for this dissertation's testing regimes. This includes large scale saturated – undrained cyclic triaxial tests. These types of tests are uncommon due to their complexity. It is believed the undrained-saturated large scale specimen test program at Michigan Tech was the most ambitious test program to date.

While it is quite difficult to monitor the volumetric behavior of soil specimens while they are being subjected to resilient modulus determinations, it is essential to determine Poisson's ratio for the materials being tested in order to understand the effects of the test variables used in the standard protocol. This required the development and manufacture of unique instrumentation that would attach to the compacted specimen while inside the triaxial chamber, and measure both lateral and axial cyclic, small strain deformations during testing.

This unique instrumentation allowed for original work regarding the stress history, strain hardening, lateral and volumetric deformations, and Poisson's ratio of engineered unbound aggregates. These topics are discussed relative to gravel/base course performance and traditional assumptions regarding small strain behavior of coarse aggregate materials. Finally a qualitative model describing the small strain behavior of unbound coarse aggregates is presented.

Sponsor: Ralph J. Hodek, Ph.D., P.E., Michigan Technological University

Postcyclic Behavior of Low-plasticity Silt

Shuying Wang

Shuying Wang, Dept. of Civil, Architectural, and Environmental Engineering, Missouri University of Science & Technology, 117 Butler Carlton Hall, 1401 N Pine Street, Rolla, MO 65401, Tel: 573-202-9945, email: sywang8208@yahoo.com

Liquefaction of low-plasticity silt has been reported during earthquakes in the recent past. Excess pore pressure builds up due to the dynamic loading and then dissipates. The postcyclic behavior of low-plasticity silt was investigated in this research for materials obtained from the Mississippi River Valley. The experimental program involved specimen preparation using a slurry consolidation approach. A special technique was developed for specimen movement, which reduced the testing program time by half. Both static and cyclic triaxial tests were conducted to confirm the ability to prepare replica specimens. In order to characterize the monotonic behavior, triaxial tests were conducted to determine the effective friction angle, critical state line, and normalized behavior. Then replica specimens were subjected to cyclic loading to develop the liquefaction curve. After full liquefaction, excess pore pressure was allowed to dissipate to achieve various reconsolidation levels. The effect of full liquefaction on the permeability and compressibility was studied. The variation in postcyclic shear strength and stiffness with reconsolidation level and the effect of apparent consolidation on shear behavior were also discussed. The critical state lines for the pre- and postliquefaction conditions were compared and found to be not parallel. After limited liquefaction, two unique conditions were tested, at no reconsolidation and at full reconsolidation. The shear strength and stiffness of the material changed significantly when the limited liquefaction reached an excess pore pressure ratio of about 0.70. The experimental program culminated with the study of the effect of plasticity on the pre- and postcyclic shear behavior. Silt-bentonite mixtures resulted in modified plasticity of the material and the transformation from a dilative to a plastic behavior were captured at relatively low plasticity (PI > 6).

Sponsor: Ronaldo Luna, PhD, PE, Dept. of Civil, Architectural, and Environmental Engineering, Missouri University of Science & Technology

Hydraulic Properties of Asphalt Concrete

Ronald Eric Pease

Ronald Eric Pease, 8229 Colfax Ave NE, Albuquerque, NM 87109, Tel: 505 828-1734, cell: 505 850-9076, email: peasekitts@comcast.net

This research has applied standard unsaturated flow models and laboratory methods common to soil analysis, to characterize the hydraulic properties of asphalt concrete. Wetting and drying water characteristic curves were measured for six asphalt concrete cores. From the water characteristic curves, it is proposed that asphalt concrete may require bi-modal sigmoid curves to represent drying. It is also proposed that asphalt concrete is a hydrophobic material during wetting and a hydrophilic material during drying. The wetting curves developed for the asphalt concrete were fit with power functions, while the drying curves follow a sigmoid curve as expected for a wettable material. Unsaturated hydraulic conductivity for wetting and drying was predicted according to Mualem's (1976) solution. The unsaturated hydraulic conductivity of three asphalt cores was measured using an original method in the laboratory. The results of the measured unsaturated hydraulic conductivities were used to support the predicted values.

The unsaturated hydraulic conductivity of the asphalt concrete during wetting was described with simple power functions; very similar to ones proposed for soils, but with a different range of exponents.

The predicted values of unsaturated hydraulic conductivity during wetting, for some of the cores, fit very well with an equation derived by Parker (1989) to describe the unsaturated hydraulic conductivity of immiscible fluids. Nieber (2000) proposed that Parker's equation could be used to describe the unsaturated hydraulic conductivity of water on hydrophobic materials. This research has shown that the asphalt concrete follows Parker's equation during wetting, which supports the predictions of unsaturated hydraulic conductivity and that asphalt concrete behaves as a hydrophobic material during wetting.

Sponsor: Dr. John C. Stormont, P.E. Professor and Chair, Civil Engineering Department, University of New Mexico

A Study of Passive Earth Pressure in Anisotropic Sand with Various Wall Movement Modes

Achmad Bakri Muhiddin

Achmad Bakri Muhiddin, YP Adhiputeri, Jl Arief Rate 15A, Makassar 90113, Indonesia, Tel: 62-411-587661, email: amuhi001@odu.edu

This study investigated the effect of anisotropy on passive pressure in sands by developing computer simulation utilizing FLAC code for plane strain condition. A series of wall movement modes was applied namely translation, rotation about a point below the wall, RBT, and rotation about a point above the wall, RTT.

From comparisons with other FLAC model in translation with isotropic material, the coefficients of passive pressure Kp were similar except for combinations of zero dilation angle (ψ), low wall friction, and high angle of internal friction (ϕ). ψ has less effect on Kp than the effect of ϕ. ψ equals ½ϕ could be used without significant effects on Kp.

When comparing simulations with anisotropic material and model wall experiment in translation, peak Kpx (Kp in x direction) from simulations were higher for loose sand, close for medium dense, and about the same for dense sand. Strains at maximum Kpx were less for loose sand, close for medium sand, and higher for dense sand. In RBT modes, Kpx were higher for low "n" (ratio of distance of center of rotation to wall height), and close for high "n". In RTT mode, Kpx were higher from simulation with low "n", and close for high "n". For all modes, points of application of resultant of lateral earth pressure at large wall displacement were practically similar. However, in the early stage of wall movement, there exist some differences.

From simulations with increasing "n" with various relative densities, Kpx for RBT and RTT reached similar maximum at "n" about 2 and 15 respectively. For simulations with various ϕ in translation, RBT (n=0), and RTT (n=0), Kp of anisotropic simulations were significantly smaller than isotropic simulations. Increasing wall high from 0.5 m to 4.0 m resulted in lower Kpx in anisotropic simulations with 13% average reduction.

Sponsor: Isao Ishibashi, Professor P.E., Ph.D., Department of Civil and Environmental Engineering, Old Dominion University

Analysis of Performance and Reliability of Offshore Pile Foundation Systems based on Hurricane Loading

Jiun-Yih Chen

Jiun-Yih Chen, The University of Texas at Austin, 1 University Station C1792, Austin, Texas 78712, U.S.A., 1-512-657-7182, jiunyih_chen@yahoo.com

Jacket platforms are fixed base offshore structures used to produce oil and gas in relatively shallow waters worldwide. Their pile foundation systems seemed to perform better than what they were designed for during severe hurricanes. This observation has led to a common belief in the offshore oil and gas industry that foundation design is overly conservative.

The objective of this research is to provide information to help improve the state of practice in designing and assessing jacket pile foundations to achieve a consistent level of performance and reliability. A platform database consisting of 31 structures was compiled and 13 foundation systems were analyzed using a simplified foundation collapse model, supplemented by a 3-D structural model.

The predicted performance for most of the 13 platform foundations is consistent with their observed performance. These cases do not preclude potential conservatism in foundation design because only a small number of platform foundations were analyzed and only one of them actually failed. The potential failure mechanism of a foundation system is an important consideration for its performance in the post-hurricane assessment. Structural factors can be more important than geotechnical factors on foundation system capacity. Prominent structural factors include the presence of well conductors and jacket leg stubs, yield stress of piles and conductors, axial flexibility of piles, rigidity and strength of jackets, and robustness of foundation systems. These factors affect foundation system capacity in a synergistic manner. Sand layers play an important role in the performance of three platform foundations exhibiting the largest discrepancy between predicted and observed performance. Site-specific soil borings are not available in these cases. Higher spatial variability in pile capacity can be expected in alluvial or fluviatile geology with interbedded sands and clays.

The uncertainties in base shear and overturning moment in the load are approximately the same and they are slightly higher than the uncertainty in the overturning capacity of a 3-pile foundation system. The uncertainty in the overturning capacity of this foundation system is higher than the uncertainty in shear capacity. These uncertainties affect the reliability of this foundation system.

Sponsor*: Prof. Robert B. Gilbert, The University of Texas at Austin*

Groundwater Inflow into Rock Tunnels

Ran Chen

Ran Chen, Ph.D., 2880 Landmark Dr., Marietta, GA, 30060, Tel: 512-769-7770, email: ranchen2010@gmail.com

Prediction of groundwater inflow into rock tunnels is one of the essential tasks of tunnel engineering. Currently, most of the methods used in the industry are typically based on continuum models, whether analytical, semi-empirical, or numerical. As a consequence, a regular flow along the tunnel is commonly predicted. There are also some discrete fracture network methods based on a discontinous model, which typically yield regular flow or random flow along the tunnel. However, I observed that in hard rock tunnels, flow usually concentrates in some areas, leaving much of the tunnel dry. The reason for this is that, in hard rock, most of the water flows in rock fractures; fractures typically occur in a clustered pattern rather than in a regular or random pattern. I develop a new method in this work that can model the fracture clustering and reproduce the flow concentration. After an elaborate literature review, a new algorithm is developed to simulate fractures with clustering properties by using geostatistics. Next, I discuss a discrete fracture network that was built and simplified. In order to solve the flow problem in the discrete fracture network, an existing analytical-numercial method must be improved. Two case studies illustrate the procedure of fracture simulation. Several ideal tunnel cases and one real tunnel project are used to validate the flow analysis. I determine that fracture clustering can be modeled and flow concentration reproduced by using the proposed technique.

Sponsor: Fulvio Tonon, Ph.D., P.E. (Texas, Italy), Assistant Professor, Department of Civil Engineering, The University of Texas at Austin

Use of Non-Steel Fiber Reinforcement in Concrete Tunnel Lining

Sang Yeon Seo

Sang Yeon Seo, GS E&C Research Institute, 417-1 Deokseong-ri, Idong-myeon, Yoingin-si, Gyeonggi-do, 449-831, Korea, email: syseo01@gsconst.co.kr, Tel: +82-10-2772-4728

Fiber reinforcement is being widely used in concrete tunnel linings these days. Using fiber reinforcement can save not only cost, but also labor and time spent on construction. However, many owners hesitate to incorporate fiber reinforcement in tunnel lining due to lack of experience with and knowledge of the behavior of fiber reinforced concrete (FRC)

In this study, fiber reinforced concrete was made with various kinds of fibers such as steel fiber, macro-synthetic fiber and hybrid fiber (a blend of macro-synthetic fiber and glass fiber). Many experimental tests were performed to investigate the compressive, flexural and shear behavior of fiber reinforced concrete. In addition to the structural capacity of FRC, the distribution of fiber reinforcement inside the concrete matrix was investigated. Test results of these experimental tests were thoroughly examined to compare and quantify the effects of fiber reinforcement. Next, the test results were used to generate axial force-bending moment interaction diagrams based on current design approaches. In addition, the current design approaches were modified to estimate the accurate and exact value of bending moment. Fiber reinforcement clearly improved the structural performance of tunnel lining. The post-peak flexural and shear strength was significantly influenced by the type and amount of fiber reinforcement.

Sponsor: Fulvio Tonon, The University of Texas at Austin

Analysis of Spatial Variability in Geotechnical Data for Offshore Foundations

Jeong Yeon Cheon

*Jeong Yeon Cheon, Ph.D, Tel: 512-786-2375,
email: luv.doogie@gmail.com*

Deep foundations, such as piles and suction caissons, are used throughout an offshore oil and gas production facility in deepwater. Ideally, the values of geotechnical properties for foundation design are determined by results from geotechnical investigation programs performed at the site of the foundation. However, the locations for facilities are not known exactly when soil borings are drilled and the footprint of a facility in deepwater can be very large with numerous foundation elements spread out over miles. Therefore, it is not generally feasible to perform a site-specific investigation for every foundation element.

The objective of this research is to assess, analyze and model spatial variability in geotechnical properties for offshore foundations. A total of 97 geotechnical investigations from 14 offshore project sites covering the past twenty years of deepwater development in the Gulf of Mexico are compiled into a database. The geologic setting is primarily a normally to slightly overconsolidated marine clay, and the property of interest for the design of deep foundations is the undrained shear strength.

The magnitude and characteristics of variability in design undrained shear strengths are analyzed quantitatively and graphically. Geostatistical models that describe spatial variability in the design shear strength properties to the distance away from the available information are developed and calibrated with available information from the database. Finally, a methodology is presented for incorporating the models into a reliability-based design framework to account for spatial variability in foundation capacity. Design examples are presented to demonstrate the use of the reliability methodology.

Based on the design undrained shear strength profiles for the past 20 years in this Gulf of Mexico deepwater area, the design undrained shear strength varies spatially but does not depend on the time or method for site investigations. There are nonlinear spatial relationships in the point shear strength laterally and vertically due to stratigraphy such that depth-averaged shear strengths are correlated over further distances than point shear strengths. The depositional forces are an important factor causing spatial variations in the undrained shear strength, with greater variation and less spatial correlation in the more recent hemipelagic deposits (about upper 60 feet) than the deeper turbidite deposits and along the shelf versus off the shelf. The increased conservatism required in deep foundation design due to spatial variability when site specific strength data are not available is generally small with less than a five percent increase required in design capacity in this geologic setting.

Sponsor: Robert B. Gilbert, Ph.D., P.E., Professor, Civil, Architectural and Environmental Engineering, The University of Texas at Austin

A Multi-Axial Tension Test for Geotextiles

Theresa Louise Andrejack

Theresa Louise Andrejack, Bucknell University, Tel: 570-577-2340, email: andrejack@gmail.com

The use of geotextiles as reinforcement is well-established in geotechnical applications. However, some uses of these materials occur in geotechnical systems where the in situ loading and boundary conditions on the geosynthetic vary greatly from laboratory testing conditions that are used to characterize their constitutive behavior. In this work, the development of a new large-diameter experimental device capable of applying multi-axial, out-of-plane loading to relatively large geosynthetic specimens (48 cm diameter) is presented. Although similar in concept to the types of apparatuses typically used for the established Multi-Axial Tension Test for Geosynthetics, this newly developed device is unique in that load is directly applied to the circular specimen using a rubber membrane, thus allowing pervious materials such as geotextiles to be tested. A key advantage of the device is that it mimics the in-service loading conditions of geosynthetics used in a range of design applications including the spanning of subsurface voids and geosynthetic-reinforced pile-supported embankments.

Constant strain rate, multi-axial tension tests were completed on a range of seven geotextiles that varied in mass per unit area, resin type, anisotropy, fiber type, fiber density, and weave. Two methods for interpreting the results from the multi-axial test, the constant-thickness and constant volume methods, are derived and compared. Uniaxial and fiber tension tests were also performed to provide a better understanding of the multi-axial test results. Three dimensional models, constructed using photogrammetry, were created to evaluate the assumptions that are used in the interpretation of the multi-axial test results. These models also provide insight into the micro-level behavior of geotextiles in multi-axial tension.

The constant strain rate, multi-axial test results indicate that there is a significant deviation in the response of geotextiles in multi-axial tension compared to their response in uniaxial tension. Although ultimate strength values were found to be comparable using the constant volume interpretation of stress, the ratio of secant modulus values from the multi-axial test over the uniaxial tension test at 2%, 5%, and 10% strain are consistently on the order of 0.6 – 0.9.

The implications of using uniaxial test parameters in analytical and numerical models where multi-axial stress is present is discussed.

Sponsor: Joseph Wartman, Ph.D., University of Washington

Deformational Behavior of Fouled Railway Ballast

Ali Ebrahimi

Ali Ebrahimi, Research Associate, Civil and Environmental Engineering, University of Wisconsin-Madison, Madison, WI 53706, email: ebrahimi.ali59@gmail.com

One of the major concerns facing the freight rail industry in the United States (US) is increased substructure maintenance due to heavier freight load and higher speed trains. The main objective of this dissertation is to characterize deformational behavior of railway ballast and predict the required maintenance of railway track due to deformation of rail substructure. A testing protocol was developed to quantify the deformation of ballast at a laboratory-scale which closely simulates field conditions. Accumulation of plastic deformation of railway ballast under traffic loading was discussed. Effect of type of fouling, fouling content, moisture, and state of stress on plastic deformation of ballast was discussed. Mechanisms, such as contaminated contact points of ballast particles and change in strength properties of fouling materials, that affect the plastic deformation of ballast were described. A geophysical (i.e., an electromagnetic surveying) technique to inspect the fouling content and moisture in railway ballast was presented. A deformation model for railway ballast was developed to predict maintenance cycles for railway track due to deformation of rail substructure.

Sponsors: Tuncer B. Edil, and James M. Tinjum, University of Wisconsin- Madison

Compression Behavior of Solid Waste

Christopher A. Bareither

Christopher A. Bareither, Research Associate, Geological Engineering, University of Wisconsin-Madison, 1217 Engineering Hall, 1415 Engineering Drive, Madison, WI 53706, Tel: 708-297-4844, email: bareither@wisc.edu

A field-scale experiment (Deer Track Bioreactor Experiment - DTBE) and series of laboratory experiments were conducted to evaluate the physical, chemical, and biological response of solid waste with leachate addition. Laboratory experiments designed to replicate field-scale conditions were conducted in 64, 100, and 305-mm diameter compression cells. Physical and biochemical mechanisms contributing to waste compression were evaluated to distinguish immediate compression, mechanical creep, and biocompression phases. Waste from the DTBE was used in all laboratory experiments; three fresh wastes and three decomposed wastes were tested. The immediate compression ratio (Cc') for the DTBE was 0.23; and Cc' ranged between 0.22 and 0.28 for all wastes in 305-mm cells. A waste compressibility index (WCI) is proposed for estimating the Cc', and is based on dry weight water content, dry unit weight, and the percent contribution of organic waste. The rate of mechanical creep (CaM') was dependent on waste composition, with larger CaM' corresponding to higher WCI and higher ratios of cellulose + hemicellulose to lignin ([C+H]/L). Leachate addition to solid waste was shown to increase waste compression via moisture-induced softening and enhancing anaerobic decomposition. During leachate dosing in the DTBE, the rate of compression varied ranged from a CaM' of 0.048 to a biocompression ratio (CaB') of 0.35. This variation was due to waste temperature fluctuations that were believed to have suppressed (at temperatures < 42 °C) and stimulated (at temperatures > 42 °C) biological activity. Laboratory-derived settlement parameters were shown to be relevant to field-scale compression behavior when contributions of physical and biochemical compression were equivalent. Application of a first-order decay rate settlement model revealed a scale effect on the elapsed time for onset of biocompression and first-order decay rate. An increase in experiment scale corresponded to an increase in time for onset of biocompression and a decrease in decay-rate.

Advisers: Craig H. Benson, PhD, PE, DGE, and Tuncer Edil, PhD, PE, DGE, University of Wisconsin-Madison

Application of Constriction Size Based Filtration Criteria for Railway Subballast Under Cyclic Conditions

Laricar Dominic O. Trani

Laricar Dominic O. Trani, BSc (Civil), MEng, PhD, MIEAust, Geotechnical Engineer, Coffey Geotechnics, 8/12 Mars Road, Lane Cove West, NSW 2066 Australia, Tel: (+61) (2) 9911 1000, Fax: (+61) (2) 9911 1001, Mobile: (+61) 406 384 657, email: dominic_trani@coffey.com

In rail track environments, the loading system is cyclic unlike the steady seepage force that usually occurs in embankment dams. The mechanisms of filtration, interface behavior, and time dependent changes of the drainage and filtration properties occurring within the filter medium require further research to improve the design guidelines. A novel cyclic process simulation filtration apparatus was designed and commissioned at the University of Wollongong, and a standard test procedure was established. The test apparatus was designed to simulate heavy haul train operations. The key parameters that influence the change in porosity and pore water pressure within the subballast layer under cyclic conditions in rail track environments were identified.

Laboratory results suggest that the present subballast selection criteria adopted by the railway industry do not address the filtration mechanism of subballasts under cyclic conditions. Subballasts containing approximately 20% fine sand and 30% fine gravel are too porous to effectively capture the fines within its voids. Laboratory findings further show that uniformly graded subballasts with particle range of 0.15 to 0.425 mm not more than 30% had an enhanced filtering capacity. Due to the lack of mechanical resistance against axial deformation, the application of cyclic stress to uniformly graded subballasts reduces porosity and enables the filter to trap migrating fines more effectively. Moreover, this intrusion of fines changes the particle size distribution of the subballast which reduces its porosity and further inhibits drainage.

A multi-layer mathematical approach was used to predict the time dependent permeability of this filter, with (a) a reduction in porosity as a function of compression under cyclic loading, and (b) the amount of fines trapped within the filter voids, being the two main aspects of this proposed model. Laboratory test results conducted on a novel cyclic loading permeameter were used to validate the proposed model. The set of equations that forms an integral part of the proposed model is then presented as compact visual guidelines anticipated to provide a more practical tool for railway practitioners.

Sponsor: Buddhima Indraratna, PhD, FIEAust, FASCE, FGS, FIES, DIC, CEng, CPEng., Professor of Civil Engineering, Head, School of Civil, Mining and Environmental Engineering; Director, Centre for Geomechanics & Railway Engineering; Faculty of Engineering, University of Wollongong

Analytical and Numerical Study of Soil Disturbance Associated with the Installation of Mandrel-Driven Prefabricated Vertical Drains

Ali Ghandeharioon

Ali Ghandeharioon, Faculty of Engineering, University of Wollongong, Wollongong, NSW 2522, Australia, email: ghandeharioon@gmail.com

Prefabricated vertical drains (PVDs) combined with preloading have gained in popularity among the most effective ground improvement techniques available to mitigate the unacceptable differential settlements caused by the heterogeneity and high compressibility of soft soil deposits. In this thesis the installation of mandrel-driven PVDs and associated disturbance in cohesive soils were studied by conducting analytical investigations, laboratory experiments, and numerical modeling.

An analytical study of mandrel penetration in soft saturated clays was carried out with a new elliptical cavity expansion theory (CET). The elliptical CET was developed using modified Cam clay parameters to address the undrained analysis that accounts for the rate of mandrel penetration and the time for predicting internal pressure in the cavity, corresponding stresses and excess pore pressure in the soil while driving the mandrel. A more realistic elliptical smear zone based on the elliptical CET was introduced while the disturbed soil surrounding the mandrel was characterized by the plastic shear strain normalized by the rigidity index.

A number of large-scale laboratory tests that incorporated the field conditions and effects of confining pressures were performed. A consolidometer specifically designed for the purpose, and a machine capable of driving mandrels at realistic rates were used in these experiments. The variations of pore water pressure during installation of a mandrel-driven PVD were monitored. The smear zone was then analyzed to establish a relationship between its size and the in-situ effective stresses.

The installation of a mandrel was simulated numerically using a commercial finite element software package, ABAQUS. The finite element models included coupled analyses with a large-strain formulation. There was agreement between the pore pressures measured in the laboratory and the finite element predictions. The extent of smear zone was studied numerically as well.

In addition, a number of case histories taken from Malaysia and Thailand were analyzed to evaluate the associated soil disturbance during installation of PVDs. The results of these analyses indicated that the developments can be applied to field conditions.

Supervisors: Prof. Buddhima Indraratna, and Dr. Cholachat Rujikiatkamjorn, University of Wollongong

Cyclic Densification of Ballast and Associated Deformation and Degradation

Pramod Kumar Thakur

Pramod Kumar Thakur, 6/111 Station Rd., Sunnybank, QLD, 4109, Australia, Tel: +61 403 702 460, email: pramodkumar.thakur@gmail.com

Railway ballast forms a major component of a conventional rail track and is used to distribute the dynamic load to the subgrade, providing a smooth running surface for the train. Heavier cyclic loadings on an existing track are inevitable due to increased demand for freight transport for mining and agriculture industries and for public transport via trains due to increased fuel costs. This loading has caused progressive deterioration and densification of ballast leading to loss of track geometry and differential track settlement. Consequently, the tracks require frequent maintenance. Understanding the behaviour of ballast under increasing load conditions is imperative for the optimum use of ballast and longevity of the maintenance period. Therefore, this research aims to investigate the densification and degradation behaviour of ballast under high frequency cyclic loading.

A series of laboratory experiments were conducted using large-scale cyclic triaxial equipment under high frequency loading. The experimental results revealed that both the densification and breakage of ballast increase with an increase in the frequency and number of cycles. While the resilient modulus of the ballast was found to decrease with increasing frequency, it increased with the increasing number of cycles and confining pressures. The Discrete Element Method (DEM) was employed to study the mechanism of particle breakage at the particle scale level. The DEM-based software, Particle Flow Code in 2-Dimensions (PFC2D), was used to simulate biaxial tests under cyclic loading. The DEM simulation results revealed that particle breakage is a governing aspect of the actual deformation behaviour of granular material. Moreover, the DEM results confirmed that the particles break under tension and that this breakage is mainly oriented and concentrated in the direction of particle movement. Finally, a cyclic densification model, based on a critical state framework and calibrated and validated using laboratory experimental results, was proposed using the continuum mechanics approach incorporating particle breakage.

Supervisors: Dr. Jayan S. Vinod & Prof. Buddhima Indraratna, Centre for Geomechanics and Railway Engineering, School of Civil, Mining and Environmental Engineering, University of Wollongong

Factors Affecting the Capacity of Open and Closed-Ended Piles in Clay

Paul Doherty

Paul Doherty, University College Dublin, Civil Engineering Department, Newstead Building, Belfield, Dubin, Ireland, Tel: 0035317163229, fax: 0035317163297, email: paul.doherty@ucd.ie

This thesis describes a series of pile tests conducted at a soft clay test bed site in Belfast, Ireland. These field tests used both highly instrumented model piles, capable of measuring the effective stresses in the surrounding soil, and full-scale uninstrumented concrete piles. Two open-ended pile installations and one closed-ended installation were performed, after which the piles were subjected to cyclic loading and rapid static loading. In addition, two concrete piles installed as part of a previous body of research were subjected to maintained load tests following a 10 year ageing period. The testing schedule was designed to assess the areas of uncertainties inherent to common offshore pile design practice in cohesive soils. These factors included: the impact of end condition, cyclic loading, rate effects and the impact of time on capacity.

An extensive review of the literature was undertaken which confirmed that offshore pile design has undergone significant advances in recent years due to a transition from total stress to effective stress design methods. The improved reliability of recent design approaches has been confirmed through comparative database studies. However the conditions offshore differ significantly from the typical database piles. The databases largely comprise small diameter closed-ended piles in contrast to the open-ended piles used offshore, resulting in a potential bias due to the end condition. In addition, the databases are largely populated by maintained load tests (MLT), whereas the in-situ loading conditions are primarily cyclic and potentially rapid due to storm events with rise times in the order of seconds rather than hours, as for MLTs. In addition, offshore structures are usually designed for events that may not occur for a significant period of time after installation and therefore the impact of time on pile capacity needs to be considered. This thesis presents an experimental investigation of these factors with a view toward incorporating these biases into the design of offshore piles.

To investigate the impact of end condition, both open-ended and closed-ended pile tests were performed. The design, development and construction of these piles were carried out such that they were representative of offshore geometries but at a reduced scale for maneuverability and testing requirements. The end condition was shown to have a significant impact on the base resistance and a lesser impact on the shaft resistance. The base resistance was seen to increase directly with the degree of plugging experienced during installation. In contrast the unit shaft resistance was relatively independent of the base geometry with the closed and open-ended piles mobilizing similar radial effective stresses during installation. The long term shaft resistance suggested minimal impact of end condition, with the open-ended pile mobilizing comparative stresses to the closed-ended pile. These results suggest that design equations formulated from closed-ended tests can be used to predict open-ended shaft capacities in normally consolidated clays.

The cyclic load tests demonstrated a marked transition from stable to unstable behaviour for loads in excess of 74% of the static capacity. Unstable behaviour was characterized by rising pore pressures and decreasing effective stresses alongside accelerating displacements. Rapidly applied CRP loads yielded capacities well above the estimated static capacities. Interface dilation was shown to control the resistance to rapid loading, with the capacity increasing dramatically in response to decreasing shear induced pore pressures, which typically occurred after 2.5-4.0 mm pile head movement. Similar depressed pore pressures were observed over the initial 10 cycles of load during unstable cycling. The implication for design is that offshore piles can withstand single rapid events in excess of the static capacity, or can withstand a limited number of high level cycles above the loading threshold but cannot sustain continuous high level cycling above approximately 74% of the capacity.

The reload testing conducted on the 250mm square concrete piles resulted in capacities which were 50% higher than the initial virgin load tests, suggesting a positive ageing effect. Comparison between these tests and the literature showed good agreement. The potential for incorporating time effects into the design process is demonstrated through a simple reliability analysis which highlights the improved reliability of piles with time.

Supervisor: Dr. Kenneth Gavin, University College Dublin

Automatic Generation of Solid Models of Building Façades from Lidar Data for Computational Modelling

Linh Truong-Hong

Linh Truong-Hong, Room G67, Newstead Building, Urban Modelling Group, School of Architecture, Landscape and Civil Engineering, University College Dublin, Belfield, Dublin 4, Ireland, Phone: +353 1716 3232, email: linh.truong-hong@ucdconnect.ie

For assessment of building damage, geometric models of existing buildings are required for analyzing processing by using advanced finite element codes. Traditionally, the geometric models were often created by using manual surveys that are expensive and time consuming, especially when numerous or complex buildings are involved. Light Detection and Ranging (LiDAR) technology provides a new technique to measure surfaces of objects with fast and high accuracy, but has to date not been effectively exploited as the basis for computational models.

In this study, the terrestrial laser scanning data were used as the input for several novel approaches to automatically reconstruct geometric models for computational modeling by employing a voxelized octree representation. Possible boundary lines of openings and the façade were determined from the boundary points underlying voxels on their boundaries. A new method called the “flying voxel” approach was devised to determine voxel position with respect to a façade and its major features. Finally, all full voxels were converted into a neutral file describing the solid model for importing directly finite element codes.

Reliability of the algorithms was verified against measured drawings based on the accuracy of the geometric models and the efficacy of their response during computational analysis. In term of the physical geometry, overall dimensions and opening areas of the building facades were respectively less than 1.5% and 3.0% of relative errors compared to real facades. An average absolute error of opening dimensions was mostly 33.4mm. In terms of building response, a macro model with non-linear analysis was adopted to analyze the buildings subjected to excavation-induced settlements. Responses deviated by less than 5% of the relative errors in angular distortion and nodal displacements compared to finite element results obtained from the real façades. The maximum estimated absolute error of nodal displacements at nodes of interest was around 4.3mm. Also, distribution of stress-strain and cracking patterns were graphically consistent with ones derived form the measured drawings.

Sponsoring Professor: Dr. Debra F. Laefer, Urban Modelling Group, School of Architecture, Landscape, and Civil Engineering University College Dublin

Instabilities in Sands

Alfonso Mariano Ramos Cañón

Alfonso Mariano Ramos Cañón, Geophysical Institute Javeriana University. Cra 7 No 42-27. Bogotá – Colombia, email: a-ramos@javeriana.edu.co

The purpose of this dissertation is to contribute to the detection of the onset of geomechanical instabilities in sands under dry/drained and saturated/undrained conditions. In order to accomplish this objective, a framework for detecting instabilities based on either a mathematical concept (i.e., loss of uniqueness - bifurcation) or physical one (i.e., Hill´s instability) is generated to derived criteria applicable to the most common instabilities: localized drained instability in dense sands (shear bands), diffuse undrained instability in loose sands (liquefaction) and diffuse drained instability in loose and dense sands (debris flow). Each one of these three instabilities is studied independently. The criteria are compared against experimental results available in the literature, and reasonable agreement is achieved. From a practical perspective, the contributions of this work expand the repertoire of potential instabilities that have been reported in case studies of puzzling slope instability failures under drained and undrained conditions.

Sponsor: Professor Arcesio Lizcano, University de Los Andes. Department of Civil and Environmental Engineering. Cra 1E No 19A – 40. Bogotá - Colombia.