Civil engineers have always been called upon to develop the infrastructural system of societies. Sometimes, this call in the modern world is extended beyond the traditional aspects of the profession because of other imposed conditions. In Singapore for example, the lack of land can be a serious barrier to the economic and social development of the city state. The need to create space to satisfy demands of society has pushed the frontier of civil engineering research and practice to a level beyond the norm. To satisfy the thirst for space, innovative ideas have been sought to utilize underground rock caverns as an alternative to above-ground land space. In order to integrate such space for everyday living and utilization, much research is necessary. To this end, the School has embarked on an extensive research program that incorporates many aspects of civil engineering disciplines to ensure a seamless integration of underground space utilization, safety and health.

In order to have an overall understanding on how underground space can be successfully integrated as part of the infrastructural system of the country, the school is presently hosting an underground technology and rock engineering research program (UTRE), which is sponsored by Defense Science and Technology Agency. The objectives of the UTRE programme are:

1. To carry out quality research of direct interest and potential applications to underground protective engineering;
2. To develop technologies related to rock engineering and underground cavern constructions;
3. To offer professional consultancies for underground space development in Singapore;
4. To conduct manpower training, technology transfer, and develop international collaborations.

The structure of the UTRE programme is shown in Fig. 1. Four major research areas are, namely, (A) Rock Dynamic Testing and Constitutive Models; (B) Modelling, Design and Construction; (C) Subsurface Infra-Structure Planning and 3D-GIS, and (D) Safety, Health and Environmental Issues and Structural Monitoring.

**A. Rock Dynamic Testing and Constitutive Models**

The objective of this research component is to investigate the dynamic properties of different rocks and to develop dynamic constitutive models that are suitable for numerical simulation of underground structures subjected to blast and/or impact load. To this end, split Hopkinson Pressure bars and gas guns...
are available in the Construction Technology Laboratory. Dynamic tests for rock specimens under high strain rate loading are tested. The pressure bars are made of steel alloy, aluminium alloy, PMMA which can be used to test various materials from very soft (for example, aluminium foam) to very strong (for example, granite). A set of granite bars are fabricated to test wave propagation across discontinuous medium. Figures 2(a)-(c) show the pressure bar and gas gun equipment in the laboratory.

Numerical models by using smoothed particle hydrodynamics are also developed (See Fig. 3). The program can be used to simulate rock splitting and crushing failure in different conditions. The material heterogeneity of rock and concrete is considered by distributing random material properties for the particles. The program has been extended to a three-dimensional model which is able to simulate the dynamic failure of rock specimens in the lab tests (Fig. 4).

B. Modelling, Design and Construction

Research area B consists of three sub-projects, namely, (B1) Development of Advanced Numerical Modelling of Underground Structures under Dynamic Loads; (B2) Rock Mechanics and Engineering for Caverns and Tunnels; (B3) Smart Sensor Technology for Underground Structural Monitoring. Project B1 mainly develops advanced computation models for simulation of rock mass. A Discontinuous Deformation Analysis (DDA) program has been developed which is able to model the rock mass breakage under explosion load (Fig. 5).

Project B2 provides technical supports to various engineering problems which act as the core of the UTRE programme and combines various research outputs from other UTRE projects. It involves site investigation, site characterisation and site selection for underground development; construction safety for underground large cavern construction; soft, weak and problematic ground tunnelling and mechanised tunneling; design of underground rock structures under dynamic loads; and optimise blasting design and support design for underground caverns.

UTRE group has assisted in the design and construction of several underground facilities in Singapore. Fig. 6(a) shows one of the chambers of the Underground Ammunition Facility (UAF) constructed by MINDEF, Singapore. Fig. 6(b) is a schematic plot on site investigation of an underground oil storage under planning.

Project B3 develops advanced techniques for Comprehensive Monitoring of Underground Structures during Construction and Operation. Fibre optical sensors and PZT patches have been applied to measure the deformations of underground structures (Fig. 7). Advantages of the new techniques by using FOS and PZT include fast dynamic response, long term durability, negligible ageing, high sensitivity, and immunity to ambient noise, etc.

C. Subsurface Infra-Structure Planning and 3D-GIS

This project concentrates its effort on two aspects: (1) to develop a framework of a subsurface 3D-GIS for Singapore
and integrating with planning. In a nation of limited space, planning is of paramount importance. The project also examines extensively the planning issue for underground facility development in Singapore. It will evaluate and plan underground strategic storage and facilities for various defense applications, study and assess the need of critical underground infrastructure and protection, and analyze the need of underground space development in general for the nation.

D. Safety, Health and Environmental Issues and Structural Monitoring

The project aims to study the science and technology of underground space utilization. It covers, most importantly the fire safety issues for large underground space utilization (Fig. 9). A fire safety guide for large underground space will be produced for Singapore.

With foreseeable increasing use of underground space, particularly for office, laboratory and leisure, it is important to understand the interaction between underground space and human. The project also examines the utilization of underground space for human activities and studies various safety and health issues, and studies psychological impact on human in an underground environment and human-environment interaction (Fig. 10).

Economic and cost issues of cavern construction are also considered. Rock tunnel and cavern construction cost data worldwide will be collected and compiled to form the basis of a cost estimate database for Singapore’s rock tunnel and cavern construction in different formations and ground conditions and different applications. The database will serve for the cost estimate of new cavern projects.
Figure 8(a) 3-Dimensional geological map of Singapore

Figure 8(b) 3-Dimensional models of NTU

Figure 8(c) 3-Dimensional geological information

Figure 9. Zone model for underground fire

Figure 10. Modeling evacuation in compartment and tunnel
RESEARCH CENTRES

Activities of Center for Advanced Construction Studies (CACS) from July 2005 to May/June 2006

- Training on “Productivity and Benchmarking in the Construction Industry” for Ministry of Education, Bhutan. Conducted over a 4-week period between 8-Jul-05 and 5-Aug-05 by A/P Ting Seng Kiong, A/P David Chew, A/P Robert Tiong, Ast/P Chen Po-Han and Ast/P Charles Cheah.

- A short course on “Fire Engineering: Design of Steel and Composite Structures” was conducted by A/P Tan Kang Hai and Prof Ian Burgess from University of Sheffield, U.K. on 25-26 July 2005.

- Adj Prof Wong Yui Cheong gave the following seminars/short courses:
  - 5 Aug 2005: Tender Exercise & Tender Evaluation of Construction Projects
  - 23 Sept 2005: An Overall View of Construction Project Procurement by Awarding a Design-&-Build Contract
  - 21 Oct 2005: Some Pointers for Better Contracting of Construction Projects

- CACS hosted a public seminar on “An Introduction to Political Risk Insurance and How Underwriters Assess Political Risk” given by Mr. Daniel Wagner from the Asian Development Bank on 13 April 2006.

- An international symposium on Advances in Steel and Composite Structures was organized on 19 May 2006. Speakers include: A/P Chiew Sing-Ping, A/P Lie Seng-Tjhen, A/P Tan Kang Hai, A/P Richard JY Liew (NUS), Prof Chung Kwok-Fai (HK), Prof Han Lin-Hai (China), Prof Li Guo-Qiang (China), Prof Brian Uy (Australia) and Prof Ben Young (HK).

- CACS is working on a NEA-NTU-JTC project entitled “Use of recycled rubber tyres in civil engineering applications”. The objective of this two-year project is to explore the use of recycled rubber tyres in civil engineering applications that adopt local materials and construction technology, and are suitable for the aggressive climatic and service conditions in Singapore and the tropical region. Focus is particularly on applications that are technically beneficial, economically viable, and non-hazardous to the environment. The scope of study includes: (a) Review of current state of knowledge to identify the use of recycled rubber tyres in viable civil engineering applications; (b) Laboratory tests to evaluate the basic engineering properties and performance of rubberized material required in each application; (c) Pilot tests and site trials to monitor the performance of the actual completed structure for each application; and (d) Provision of guidelines for the use of recycled rubber tyres in civil engineering applications under severe tropical climates and service conditions.

Activities of Environmental Engineering Research Centre (EERC) from July 2005 to May/June 2006

Public Seminars / Lectures

- Natural Wetlands and Wetland Invasive Plant in China
  Dr. He Chi Quan UO’ihQ; School of Environmental and Chemical Engineering
  Shanghai University; P. R. China
  24 Mar 2006

- Basin Scale Phenomena in Stratified Lakes
  Professor Jörg Imberger; Director, Centre for Water Research and Vice-Chancellor’s Distinguished Fellow, University of Western Australia
  10 May 2006

- Republic Polytechnic Seminar: Latest Technology on Conservation
  RP Conservation Week, Guest Seminar
  A/Prof Wang Jing-Yuan, NTU
  4 July 2006

- Preparing a Manuscript for Successful Publication
  Presented by CEE Faculty Staff:
  Prof. Chiew Yee Meng: Editor’s view on manuscripts submitted for publication
  Prof. Cheng Nian Sheng: Tip on successful publication (Water Resource)
  Prof. Liu Yu: Tip on successful publication (ENV)
  Dr. Olena Stabnikova: Experimental design and publication
  Ms. Li Shu Yun: Tip on competitive journal paper writing
  25 Aug 2006
• **Soil Remediation Technologies: An overview**  
  Professor Ravi Naidu Foundation Professor of Environmental Remediation  
  University of South Australia  
  6 Sep 2006

• **An introduction to the Centre for Environmental Technology and Engineering, Massey University, New Zealand**  
  Dr Andy Shilton; Director of the Centre for Environmental Technology and Engineering  
  Massey University, New Zealand  
  21 Sep 2006

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**External Short Course**

• **Air Pollution Management for the Singapore Environmental Institute, National Environmental Agency (NEA)**  
  A/P Lawrence Koe; Assoc. Professor, School of CEE, NTU  
  7-8 Mar 2006

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**Conference**

• **Young Researchers Conference (YRC) 2006; 24-26 May 2006**  
  The conference hosted by EERC at the NEC, on NTU campus, is under the International Water Association (IWA), and is for young researchers (post-graduate students and researchers).

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**Colloquium**

• **An EERC-IESE Colloquium** organized by EERC specially for researchers from both EERC and IESE; 1 Sep 2006

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**Community Event**

**Clean & Green Week 2006** – This is an annual event organized by NEA at the Republic Polytechnic, 5 Nov 2006 (Sunday).

EERC supported the event in the following areas:

**Talks**

  **Youth Forum:** (Climate Change)  
  Speakers: A/P Ho Hiang Kwee (NTU, School of MAE); Dr Joel Swisher, Rocky Mountain Institute; Fabian Foo, WWF Singapore

  **Talk 1:** Waste to Energy – A Sustainable Alternative Energy Source by A/P Wang Jing-Yuan

  **Talk 2:** Water Reclamation Using Membrane Bioreactor Technology: Some Key Research Issues by A/P Darren Sun

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**Oversea Programme**

• **The 4th UNEP meeting of the UNEP Asia-Pacific Regional University Consortium**  
  Griffith University, Nathan, Brisbane, Australia; 2 – 5 Apr 2006  
  Representative: A/P Wang Jing-Yuan

• **The 5th UNEP RUC and 3rd Asian-Pacific Leadership Programme for Sustainable Development** Tongji University, Shanghai, China; 16 – 23 Sep 2006  
  Representative: A/P Wang Jing-Yuan

**Visitors**

• **Visit by Professor David Sedlak**, Professor of Civil & Environmental Engineering, University of California, Berkeley, USA; 17 Jan 2006.

• **Visit by Dr Harold Raveche**, President, Stevens Institute of Technology, USA; 22 Feb 2006.

• **Visit by a delegation from LIEN Foundation**; 27 Mar 2006.

• **Visit by Professor Thommy Svensson**, The Director of Swedish School of Advanced Asia-Pacific Studies (SSAAPS) and Mr Roger Svensson, Managing Director, The Swedish Foundation for International Cooperation in Research and Higher Education (STINT); 10 April 2006.  
  Their trip was to follow up on the Swedish delegation’s visit last November. The purpose of this visit is to meet key people at NTU to discuss two issues:
Possibilities to set up a competitive exchange programme for qualified NTU-students at master and PhD levels linked to the best Swedish universities.
Possibilities to identify interests and mechanisms for initiating bilateral research projects in the fields of bioscience, environment and water technology, and digital and interactive media. They also hope to set up joint funding arrangements between a consortium of Swedish five foundations and the NRF.

- Visit by a delegation led by Mr Xiang Huaicheng; 11 April 2006
  The former Minister of Finance, PRC, visited EERC Annexe at RTP.

- Visit by a team of researchers led by Dr Stephen Lupton (Solutions Development Manager, Honeywell Technology Center) Glen Allison (Operations Manager, Honeywell XCEED) Ong Cheng Hock (Director, Environmental Solns & Svcs, Honeywell Process Solns Asia-Pac) from Honeywell Environmental Group; 21 Apr 2006
  It is a feasibility study trip on a possible 10 man R&D research centre in Singapore. They were here to visit research centres as well as universities specializing in environmental-related R&D.

- Visit by Dr Brid from the School of Biotechnology, Dublin City University; 8 May 2006
  The visit is to explore possible collaboration on Biogrannulation and Biofilm Technologies between the two Universities.

- Visit by faculty members from Chulalongkorn University, Thailand; 11 May 2006
  The school hosted a tea reception to interact with the 21 visitors from Chulalongkorn University. Prof Show K Y represented EERC at the function.

- Visit by Mr Lee Collins from the University of Newcastle Upon Tyne; 27 June 2006
  Visit is to discuss on the possibilities of working together on Marine Energy Systems, improving reliability, availability and maintainability with respect to savings in fuel and other marine environmental related issues.

- Visit by The International Physics Olympiad (IPhO); 12 July 2006
  It was an international physics competition for pre-university and high school students. Singapore has the honor to host the 37th International Physics Olympiad (IPhO2006) from 8-17 July 2006. On July 12, 800 people (400 Olympians and 400 accompanying persons) from all 86 countries attended the NTU research centre tour.

- Visit by B&V Water Specialists; 13 July 2006
  Three of Black & Veatch’s global water specialists: Jonathan Clements (water treatment chemistry); Don Ratmayaka (water treatment processes); James Currie (Regional Operations Manager) met up with the professors of the EWRE Division to discuss potential opportunities for collaboration between B&V and NTU.

- Visit by Hanoi Water Resources University (HWRU); 24 July 2006
  The visitors were Associate Professor Dr Le Dinh Thanh, Vice Rector; Dr. Trinh Minh Thu, Vice Dean of Geotechnical Engineering; Mrs Pham Hong Nga, Vice Head of Science-Technology and International Cooperation Dept.

- Visit by Prof. Eilon Adar, Director of Zuckerberg Institute for Water Research (ZIWR) at the Ben-Gurion University, Israel; 26 July 2006
  The visit program included meeting with the faculty members of the EWRE Division, followed by lab tours.

- Visit by EWI team; 14 Aug 2006
  Meeting was attended by the Prof Ng and the EWI team, IESE management, EERC and CEE faculty members. The visit program included Lab tours to CEE Envlab, EERC-Annexe at RTP and also IESE facilities.

- Visit by Mr. K. Suzuki and Prof. Mario Tabucanon, UN University representatives; 14 Aug 2006
  Mr. Suzuki is a senior fellow of UNU’s IAS (education for sustainable development programme), while Prof. Tabucanon was former AIT President (currently on his sabbatical leave with UNU and UNE. They were in Singapore to meet with NEA and SEI (Singapore Environmental Institute) for setting up a regional centre of expertise (RCE) – Singapore on sustainable development. Director of EERC has invited them to visit NTU/EERC facilities and meet up with Dean of school and faculty members.

- Visit by a delegation from UIUC (University of Illinois at Urbana-Champaign): 18 Aug 2006
  The delegates include Prof Jesse Delia, Executive Director of International Research Relations; Dr Asghar Mirarefi, Director of International Research Partnerships. There was a Joint presentation by IESE/EERC at IESE. The delegate was interested to find out about the work in water technologies at NTU. Their goal is to identify areas of collaboration with UIUC.

- Visit by Vice-Chancellor Professor Alan Robson and Professor Imberger of the University of Western Australia; 21 August 2006
  The meeting was followed by a lab tour to introduce EERC research facilities.

- Visit by Prof Gerhard Schmitt, VP Development of ETH visited to NTU; 7 Sep 2006
  A tour to EERC-Annexe was conducted for Prof Schmitt. Purpose of the visit is to scout out NTU in preparation for Oct 9-11 visit by a senior team from ETH on 8-11 Oct led by Prof Dr Alexander JB Zehnder, as a part of the NRF grand scheme of bringing ETH/EPFL to Singapore. The objective of the visit is to interest ETH faculty in establishing a research centre in Singapore as a platform for closer and more strategic collaboration with our institutions.
• **Visit by Professor Ravi Naidu, FSSSA, FASA, FNZSSS, MRAIC C Chem**; 6 Sep 2006
  Prof Ravi is the Managing Director of the Cooperative Research Centre for Contamination Assessment and remediation of the Environment (CRC~CARE). Prof Ravi was also invited to give a public seminar at EERC during his visit to NTU.

• **Visit by US Federal Aviation Administration’s Office of Energy and Environment**, hosted by EDB; 15 Sep 2006
  The FAA visitors were Mr Carl Burleson, Director of Office of Environment and Energy; Dr Lourdes Maurice, Chief Scientific and Technical Advisor for Environment; Ms Diana Lee, FAA (based in Singapore) and Yip Beng Hoe, Senior Officer, Logistics & Transport, EDB.

• **Visit by Dr Andy Shilton**, Director of the Centre for Environmental Technology and Engineering, Massey University, New Zealand; 21 Sep 2006
  Purpose of his visit is to learn more about EERC waste to energy work and to look for international collaborators. EERC has in turned organized a public seminar for him to present the research work undertaken by his centre.

• **Visit by Sandia National Labs**; 27 Sep 2006
  There was three presentations on water related work by EWRE/EERC/EWT Cluster (A/P Edmond Lo); IESE presentation; and Water security ideas (A/P Ser Wee).

• **Visit by Swiss Delegation**; 9 Oct 2006
  The delegate was from ETH, EPFL, EAWAG and Paul Sherrer Institute by invitation of the NRF. The school was invited to give presentations on Environmental & Water Research in CEE and Water Technology-Membrane were by A/Prof Edmond Lo and Darren Sun respectively followed by a lab tour to EERC-Annexe led by A/P Wang JY.

• **Visit by French Water Technology Professor**; 30 Oct 2006
  Prof. Thevenot, a prominent expert on water issues from France was in Singapore to conduct workshops and deliver conferences as well as to meet researchers in this field. Director, EERC together with faculty staff hosted a meeting for the visitor.

• **Visit by Hwa Chong Hwa Chong Institution Science Research Centre**; 30 Oct 2006
  The visitors were briefed on EERC research work followed by tour to Environment laboratory.

• **Visit by RAadm Lui Tuck Yew, Minister of State and Mr Masagos Zulkifli, Senior Parliamentary Secretary, Ministry of Education**; 3 Nov 2006
  The visit includes a tour to Environmental Engineering Research Centre led by A/P JY Wang, Director, to showcase Research Projects at EERC.

• **Visit by GE Water CTO Ramesh**; 7 Nov 2006
  GE Water CTO was in Singapore to attend the EWT Exco International Advisory Panel meetings. He has arrived earlier to hold discussions with the various parties in Singapore including NTU and PUB. The agenda has revolved around (1) possible R&D collaborations with NTU and (2) hiring of NTU graduates to staff the 120-researcher Water R&D Centre. There was also a short discussion with Prof Tony Fane on membrane-related research.

• **Visit by Georgia Institute of Technology**; 7 Nov 2006
  EDB has arranged for a group of researchers (Jaehong Kim, Spyros Pavlostathis, F. Michael Saunders and Ching-Hua Huang) from the Environmental Engineering department of Georgia Institute of Technology to meet with CEE and EERC management. Georgia Tech has been ranked consistently in the top 10 for environmental engineering in the U.S and they would like to scope out possible education and research activities in Singapore. NTU has been invited to give the visitors an overview of its research focus and tentatively there could be potential opportunities for collaboration in research in Singapore.

**Activities of Maritime Research Centre (MRC) from July 2005 to May/June 2006**

**Research Projects and Activities:**

- Key Aspects of a LNG Terminal Construction, 29 April, 2005, NTU
- 2nd Joint MPA-NTU-NUS Industry Seminar, 28 July, 2005, Nanyang Executive Centre (NTU)
- Executive Programme on Shipping & Port Marketing, 12 - 17 September, 2005, NTU Executive Centre (Orchard Road)
- New IMO Convention on Ships’ Ballast Water Management: Need for Creative Engineering and Global Partnerships, 1st March, 2006, NTU
- Marine Engineering - Ship Repair: A presentation of NTU expertise to EDB, Singapore. 3 Aug 2006, NTU.
- Maritime Seminar Series @ NTU – Shipping. 30 Aug 2006, NTU
- Short Course on “Introduction to Maritime Economics”, Nanyang Executive Centre, NTU, 2 Sep 2006.
Activities of Protective Technology Research Centre (PTRC) from July 2005 to May/June 2006

1) Seminars

- “Tsunamis, Disasters and Defense Works” by Prof Nobuo Shuto, An International Authority in Tsunamis and Coastal Engineering, 3 May 2005

- “When and Where will be the Next Big Sumatra Earthquake? – The Burning Question” by Professor Pan Tso-Chien, Director of PTRC, 20 May 2005

- “Some Problems in Rock and Soil mechanics” by Professor Li Shihai, Vice Director of Engineering Sciences Division, Institute of Mechanics, Chinese Academy of Sciences, China, 15 July 2005

- “Protection Against Projectile Penetration” by Dr Li Qingming, Lecturer, Department of Mechanical, Aerospace and Manufacturing Engineering, UMIST, UK, 19 July 2005

- “Environmental Geotechnology Research in Department of Earth Sciences” by Professor Li Xiaozhao, Department of Earth Sciences of Nanjing University, China, 27 July 2005

- “Performance-Based Earthquake Engineering: Assessment of Seismic Risk and Financial Loss” by Dr Rajesh Dhakal, Senior Lecturer, Department of Civil Engineering, University of Canterbury, New Zealand, 24 November 2005

- “Performance-Based Seismic Design and Retrofit of R.C. Structures: Emerging Trends and Continuum Challenges” by Stefano Pampanin, Senior Lecturer, Department of Civil Engineering, University of Canterbury, New Zealand, 25 November 2005

- “Damping Models for Time-History Structural Analysis” by Dr Athol Carr, Reader in Civil Engineering, University of Canterbury, New Zealand, 29 November 2005

- “Shake Table Testing Studies of Unreinforced Masonry Walls” by Professor Carlos Ventura, Director of Earthquake Engineering Research Facility, The University of British Columbia, 8 December 2005

- Research Seminars by researchers of the Underground Technology and Rock Engineering (UTRE) Programme, 17 May 2006

2) Short Courses

- “3DGIS” by Mr Zhang Yetian, Software Specialist from LIESMARS, Associate Professor Tor Yam Khoon, Division of Ge & Tr, CEE, and Ms Zhang Xianhui, CEE PhD Student, 27 to 30 June 2005

- “UDEC Modelling” by Dr Chen Shougen, Research Scientist at CSIRO, Australia, 21 to 22 July 2005
• “Plume, Zone and CFD Modelling of Fires” by Professor Vasily B. Novozhilov, Chair in Fire Dynamics, School of Built Environment, University of Ulster, United Kingdom, Associate Professor Tan Kang Hai and Associate Professor Law Wing-Keung, Adrian, School of CEE, NTU, 17 and 18 July 2006


3) Symposium

The 4th International Symposium on “New Technologies for Urban Safety of Mega Cities in Asia” was held at the Nanyang Executive Centre on 18 and 19 October 2006. Co-organised by PTRC and the International Centre for Urban Safety Engineering of the Institute of Industrial Science, The University of Tokyo, the symposium was attended by about 100 participants from 12 countries. The participants included engineering researchers, defence science researchers, design engineers and members of the ministries. In addition, 2 keynote, 5 plenary and 63 technical papers were presented.

Professor Lim Mong King, Dean of College of Engineering of NTU, was the symposium’s Guest-of-Honour.

4) Presentations to visitors of NTU and DSTA

From 1 May 2005 to 15 September 2006, PTRC has made presentations to the following visitors of NTU and DSTA.

• Delegation from Woods Hole Oceanographic Institution (USA), to NTU on 16 January 2006

• Visit by Dr Harold Raveche, President of Stevens Institute of Technology (USA), to NTU on 22 February 2006

• Visit by Dr James Tegneli, Director of Defence Threat Reduction Agency (USA), to DSTA on 20 March 2006

5) Visitors to the Centre

The Centre has hosted the following visitors from 1 May 2005 to 15 September 2006:

• 30 June 2005. Visit from the US Defense Threat Reduction Agency (DTRA) and the Defence Science & Technology Agency (DSTA), Singapore

• 14 July 2005. Visit by the Indian Institute of Technology Kanpur & Tandon Consultants, New Delhi, India

• 26 July 2005. Visit from the U.S Army Armament Research, Development and Engineering Center (ARDEC), the US Ambassy and the Defence Science & Technology Agency (DSTA), Singapore

• 04 October 2005. Visit by Dr Tan Kim Siew, Permanent Secretary (Defence Development), MINDEF

• 19 November 2005, Visit by Professor Anne Kiremidjian, Department of Civil and Environmental Engineering, Stanford University

• 20 November 2005. Visit by Dr David Vaughan, Principal, Applied Science Division, Weidlinger Associates, Inc., USA

• 12 January 2006. Visit from the Ministry of Defence, Sweden

• 23 May 2005. Visit by 39 participants from the Defence Research & Development Seminar held at NTU

• 2 June 2006. Visit by Professor Geoff Mays, Deputy Principal, Defence College of Management and Technology (DCMT), Cranfield University, UK

• 7 June 2006. Visit by Mr Phillip Winter, Section Head, Weapons System Division, DSTA Australia

• 12 July 2006. Visit by Participants of the International Physics Olympaid (IPhO)
RESEARCH ACHIEVEMENTS

AWARDS

The research team from the Protective Technology Research (PTR) of the School was awarded the Defence Technology Prize 2006 Team (R&D). The team consists of A/P Li Bing, A/P Lok Tat Seng, A/P Lu Yong and Professor Pan Tso-Chien (Team Leader). The award is given by the Ministry of Defence, Singapore in recognition of their outstanding contributions to the defence and security of Singapore.

EDITORSHIP

A/P Li Bing is appointed as an Associate Editor of the Journal of Structural Engineering, American Society of Civil Engineers (ASCE).

A/P Tommy Wong is appointed as a Corresponding Editor of the Journal of Professional Issues in Engineering Education and Practice, American Society of Civil Engineers (ASCE).

INVITED LECTURES

A/P Chu Jian was invited to deliver lectures at the following conferences:

(1) Invited Lecture, “Strain softening and instability behavior of sand under plane-strain conditions.” International Symposium on Advances in Laboratory Testing of Geomaterials, 3 June 2006, Hong Kong, organized by Hong Kong Geotechnical Society, HKIE, HK Polytechnic University and TC29.


(6) Invited Theme Lecture, “Applications of innovative ground improvement techniques to road constructions.” 1st International Road and Traffic Conference, 7-8 July, 2005, Seoul, South Korea, organized by The Korea Transport Institute, Korea Toad & Transportation Association and International Road Federation.
## RESEARCH PROJECTS

<table>
<thead>
<tr>
<th>Project Titles</th>
<th>Principal Investigators</th>
</tr>
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<tbody>
<tr>
<td>Slope Instrumentation for the Study of Rainfall-induced slope failures in Singapore</td>
<td>Harianto Rahardjo (<a href="mailto:chrahardjo@ntu.edu.sg">chrahardjo@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Assessment of Climatic factors on slope stability. Joint collaboration with HSR University of Applied Science Rapperswil, Switzerland</td>
<td>Harianto Rahardjo (<a href="mailto:chrahardjo@ntu.edu.sg">chrahardjo@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>PUB-NTU Joint Collaboration on “Water Quality Monitoring, Modelling and Management for Catchment and Reservoir System”</td>
<td>Lo Yat-Man, Edmond (<a href="mailto:cymlo@ntu.edu.sg">cymlo@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>TEC Project. Conversion of Food Waste into Biogas &amp; Fertilizer Pilot Plant</td>
<td>Wang Jing-Yuan (<a href="mailto:cjywang@ntu.edu.sg">cjywang@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Explosion induced ground motion monitoring (Mindef-NTU-JPP)</td>
<td>Pan Tso-Chien (<a href="mailto:cpan@ntu.edu.sg">cpan@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Experimental study on rock cover damage induced by underground explosion (Mindef-NTU-JPP)</td>
<td>Ma Guowei (<a href="mailto:cgwma@ntu.edu.sg">cgwma@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Experimental investigation on fire resistance of axially-restrained RC columns under elevated temperature (Mindef-NTU-JPP)</td>
<td>Tan Kang Hai (<a href="mailto:ckhian@ntu.edu.sg">ckhian@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>MPA-NTU Joint Collaboration – Maritime and Offshore Technology R&amp;D (MRC)</td>
<td>Tan Soon Keat (<a href="mailto:ctansk@ntu.edu.sg">ctansk@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>TEC Project. Nanostructures Photocatalyst for Membrane fouling control. Collaboration with Stanford Univ, US</td>
<td>Darren Sun Delai (<a href="mailto:ddsun@ntu.edu.sg">ddsun@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Laboratory Investigations of Dynamic Mechanical Response of Shape Memory Alloys at High Strain Rates (Mindef-NTU-JPP)</td>
<td>Ma Guowei (<a href="mailto:cgwma@ntu.edu.sg">cgwma@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>The establishment of a real time control, alarm and management system for Marina Bay. PUB-CWR-NTU</td>
<td>Lo Yat-Man, Edmond (<a href="mailto:cymlo@ntu.edu.sg">cymlo@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Feasibility study of harvesting rainwater in selected areas in Tuas and Jurong West</td>
<td>Tan Soon Keat (<a href="mailto:ctansk@ntu.edu.sg">ctansk@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Safety and Risk Assessment of offshore structures containing cracks and defects</td>
<td>Lie Seng Tjhen (<a href="mailto:cstlie@ntu.edu.sg">cstlie@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Finite Element analysis (FEA) code checking as per rules on marine classification societies</td>
<td>Ma Guowei (<a href="mailto:cgwma@ntu.edu.sg">cgwma@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Low speed manoeuvring of an Ogive cylinder using pulsating jets</td>
<td>Law Wing-Keung, Adrian (<a href="mailto:cwklaw@ntu.edu.sg">cwklaw@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>Autonomous verification and validation for simulation modeling</td>
<td>Yang Yaowen (<a href="mailto:cyyang@ntu.edu.sg">cyyang@ntu.edu.sg</a>)</td>
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<tr>
<td>Use of copper slag as a land reclamation fill material in Singapore Phase 1</td>
<td>Lim Teik Thye (<a href="mailto:cttlim@ntu.edu.sg">cttlim@ntu.edu.sg</a>)</td>
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<tr>
<td>Consideration of barge flexibility in loadout operation</td>
<td>Ma Guowei (<a href="mailto:cgwma@ntu.edu.sg">cgwma@ntu.edu.sg</a>)</td>
</tr>
<tr>
<td>An innovative Multi-Chamber Airlift Biocarbon Reactor for Enhanced Pharmaceutical Wastewater Treatment</td>
<td>Liu Yu (<a href="mailto:cyliu@ntu.edu.sg">cyliu@ntu.edu.sg</a>)</td>
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<tr>
<td>Project Title</td>
<td>Lead Investigator</td>
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<tr>
<td>SEP Project. Interactive AR-Integrated System for Efficient Indoor Evacuation</td>
<td>Chen Po-Han</td>
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<tr>
<td>SEP Project. Study of Rainfall – Induced Slope Failures</td>
<td>Harianto Rahardjo</td>
</tr>
<tr>
<td>SEP Project. Integrated Analysis Procedure for Fatigue Strengths of Tubular Joints under Complex Loading Conditions</td>
<td>Lee Chi King</td>
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<tr>
<td>SEP Project. Database for Dynamic Properties of Singapore Soils</td>
<td>Leong Eng Choon</td>
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<tr>
<td>Development of Advanced Research in Civil &amp; Structural Engineering</td>
<td>Pan Tso-Chien</td>
</tr>
<tr>
<td>Molecular Basis of Formation of Active but Non-Culturable Environmental Bacterial Pathogens. (Collaboration with Curtin University, Australia)</td>
<td>Gin Yew-Hoong, Karina</td>
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<tr>
<td>Illustration Fouling Mechanism of Ultrafiltration Membrane in Water Treatment System</td>
<td>Darren Sun Delai</td>
</tr>
<tr>
<td>Catastrophic Risk Analysis of Infrastructure Systems</td>
<td>Charles Cheah Yuen Jen</td>
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<tr>
<td>Finite Element Modelling of Precast Hybrid Steel-concrete connections under seismic loadings</td>
<td>Yip Woon Kwong</td>
</tr>
<tr>
<td>Development of Nano-structured photocatalysts for membrane water treatment</td>
<td>Darren Sun Delai</td>
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</tbody>
</table>
Regulatory Framework on Foreign-invested Construction Enterprises in China

Ding Li (ding0006@ntu.edu.sg)
Chew Ah Seng David (caschew@ntu.edu.sg)

INTRODUCTION

With China’s accession to WTO in 2001, a large number of laws and regulations governing Chinese construction industry have been changed to meet the requirements of WTO and fulfill China’s commitments. Foreign contractors carrying out business activities under such an evolving legal framework will have to understand the local construction regulations and adapt to them if they want to succeed in China. However, a number of foreign construction firms opined that new regulations established post-WTO were rather restrictive and controversial (Kosmiatz, 2004). In addition, policy risks and stability of the legal system in China are among the first concerns of foreign contractors (Shen et al., 2001). To better understand the construction laws and regulations governing foreign-invested construction enterprises (FICEs) in China, this research aims to provide a comparative study on the major construction regulations governing FICEs in China. Major aspects of the construction legal system on FICEs are first identified and then compared with those in Singapore, India, and Brazil to derive their experiences in regulating foreign construction enterprises.

CONSTRUCTION REGULATORY FRAMEWORK GOVERNING FICES IN CHINA

Under China’s Ministry of Construction (MOC), the current construction legal framework consists of four levels: Laws, Administrative Regulations, Departmental Rules and Local Regulations. They are issued respectively by the National People’s Congress and its Standing Committee, the State Council, the MOC, and the local governing authorities. As shown in Table 1, the current major Chinese construction laws and regulations on FICEs are included in the level of Departmental Rules and Relevant Measures and Circulars. In this research, the comparison is only confined to those regulations on FICEs listed in Table 1, although FICEs also have to comply with other general construction laws and regulations governing both domestic firms and foreign firms.

A COMPARATIVE STUDY OF THE CHINESE CONSTRUCTION LAWS AND REGULATIONS ON FICES BEFORE AND AFTER ITS WTO ENTRY

To meet the requirements of WTO, the Chinese government promulgated the 2002 Regulations on Foreign-invested Construction Enterprises, which abrogated the regulations in 1994 and 1995. The 2002 regulations are the most comprehensive documents on administration of FICEs in the existing legal framework in China. Since then, four major supplemental regulations and measures were published (see Table 1) to incorporate necessary amendments to the 2002 Regulations. This comparison is made between the major construction regulations on FICEs before and after the WTO accession. The major aspects of changes in construction legal system on FICEs are presented below.

Entry Modes of Foreign Construction Companies
In the 2002 Regulations, the term “foreign-invested construction enterprises” includes a wholly foreign-owned construction enterprise (WFOE), a Sino-foreign equity joint venture (JV) or a Sino-foreign co-operative construction enterprises.

<table>
<thead>
<tr>
<th>Level</th>
<th>Titles of Laws and Regulations (Decree No.)</th>
<th>Date of Promulgation</th>
<th>Date with Effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Departmental Rules</td>
<td>Regulations on Administration of Foreign-invested Construction Enterprises (113)</td>
<td>27-09-2002</td>
<td>01-12-2002</td>
</tr>
<tr>
<td></td>
<td>Supplementation on Regulations on Administration of Foreign-invested Construction Enterprises (121)</td>
<td>19-12-2003</td>
<td>01-01-2004</td>
</tr>
<tr>
<td>Relevant Measures and Circulars</td>
<td>Implementation Measures on Administration of Skill Qualifications in Regulations on Administration of Foreign-invested Construction Enterprises (73)</td>
<td>08-04-2003</td>
<td>08-04-2003</td>
</tr>
<tr>
<td></td>
<td>Circular on Administration of Foreign Enterprise Skill Qualifications for Contracting Construction Works within the Territory of China (193)</td>
<td>28-09-2003</td>
<td>28-09-2003</td>
</tr>
<tr>
<td></td>
<td>Circular on Administration of Skill Qualifications of Foreign-invested Construction Enterprises (159)</td>
<td>06-09-2004</td>
<td>06-09-2004</td>
</tr>
</tbody>
</table>

Table 1 Chinese Construction Laws and Regulations on FICEs as in 2006

Source: Abstracted from the Website of Ministry of Construction, China (www.cin.gov.cn)
According to the 1995 regulations, wholly foreign-owned enterprises were not permitted within the territory of People’s Republic of China. Furthermore, establishing a Chinese corporate entity is a prerequisite for any foreign construction firms which intend to carry out projects in China. Foreign contractors can no longer undertake construction works on a project-by-project basis as they did under the old 1994 regulations.

**Classification and Registration System for Establishment and Skill Qualification**

A detailed procedure for FICEs to apply for the approval of establishment and skill qualifications was first introduced in the 2002 Regulations. The skill qualifications of construction enterprises can be classified into three categories—main contractor, special contractor and labor subcontractor, with different grades to demarcate the qualifications in each category.

As for the required capital to register the construction company, FICEs are accorded the same treatment as domestic construction firms. In the new regulations, capital contribution of the Chinese party shall not be less than 25% of the total registered capital in a Sino-foreign Equity Joint Venture (EJV) or a Sino-foreign Co-operative Joint Venture (CJV). Additionally, after the supplemental measures were promulgated, the track record of the FICE outside China can be considered in its application for the Skill Qualification Certificate (SQC).

**Restrictions on Projects Undertaken by FICEs**

According to the 2002 Regulations, the scope of works undertaken by Wholly Foreign-owned Construction Enterprises (WFOCEs) was restricted to four categories of projects. Nevertheless, a Sino-foreign EJV or a Sino-foreign CJV can carry out any projects within the scope of their respective skill qualifications without the above limitation.

In terms of entry modes, foreign contractors have to register a new corporation as the local entity before commencing projects in China. In Singapore, foreign contractors can simply proceed with their entry registration to the relevant authority. Besides registering as the local corporation, Brazil and India also provide foreign investors several other options, including the project-based office. This is however not allowed in China.

In respect of registration system and requirements, foreign participation in China is limited by the comparatively high requirements in aspects of track record, registered capital, and personnel qualifications. Among the four countries, Brazil is the only one that requires no minimum registered capital for different types of company entities. Classification system for construction companies is more complicated in Singapore and China than that in Brazil and India. In addition, China is the only country that restricts the scope of projects undertaken by WFOCEs.

**CONCLUSIONS**

Currently, China’s construction market has not fully opened to the outside world, as time is needed for the country to gradually eliminate the restrictions on foreign investment in the industry. This comparative study is aimed at identifying potential legal aspects that affect the business of foreign contractors in China. Currently questionnaire survey and interviews are being undertaken to obtain FICEs’ perspectives on the latest construction legal framework governing their operation. Data collected will be analyzed to determine the impact of the legal framework on FICEs. Barriers in construction laws and regulations on foreign entry and their business in China will be identified and suggestions will be made to improve the existing construction legal framework to further attract and oversee FICEs in China.

**REFERENCES**


Organizational Learning in International Construction Firms

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INTRODUCTION

The world is changing at an accelerating pace and some of these changes are bold and unprecedented. In the context of the volatile and complex new environment, organizations have to undergo a revolutionary, organizational change towards learning organization. International construction firms are continuously entering new geographical markets and offering a wide range of multi-disciplinary services, of which the actual construction process may only form a small part [1]. The construction industry has also become more knowledge intensive than ever before. International construction firms need to develop their capacities for learning as organizational learning capabilities enable them to continuously adapt to external and internal changes and maintain their competitive sustainable advantage in the fast changing new environment.

ISSUES THAT INTERNATIONAL CONSTRUCTION FIRMS ARE FACING

Immediate and critical issues that all international construction firms have to face are: How to develop their business capabilities to overcome market shifts and grow nationally and internationally? What are the necessary changes that international construction firms shall adapt to ensure their success in the competitive business environment? Organizational learning is the only way to achieve these objectives. Organizational changes that enable effective organizational learning are the most critical issue that international construction firms should pay attention to and take immediate action.

Although organizational learning and changes in management culture and practices have been a recurrent theme in recent years, there is comparatively little research available that explores the issues and challenges international construction firms are facing. The construction industry is widely regarded as being lacking in terms of management and technology innovations [2]. According to the research conducted by Construction Industry Institute (CII), learning organizations in the construction industry are still in the developmental stage. So exploring the barriers that obstruct construction firms from moving towards learning organization is currently the most important task world-wide.

OBSTRUCTIONS FOR CONSTRUCTION FIRMS IN DEVELOPING LEARNING CAPABILITY

Construction business environment is commonly considered as complex and dynamic. Through a pilot-study with experienced industrial practitioners and literature review, the following barriers against organizational learning, which are linked to the construction business characteristics, are identified:

- **Project-based construction business**
  Construction firm’s performance and profit are driven by project success. While most firms are content to stay afloat one or a few projects at a time, the importance of corporate-level management issues is often downplayed and organizational learning is often ignored.

- **Highly fragmented operational nature**
  Some progressive firms promote and encourage various types of learning. However, the geographical dispersion of projects often weakens the organizational learning capability and independent project management teams are often left out of headquarters’ control.

- **Uniqueness of individual project**
  Uniqueness of projects in terms of design and location implies project-specific planning and construction methods which make it impossible to standardize the process or directly apply any project knowledge to other project operations; and different risks / uncertainties may be incurred by the shift of project location and site condition. Such conditions make project staff reluctant to learn across projects.

- **Dynamic nature of project design and construction**
  Changes in construction projects are commonly made to refine design, to suit construction technology/ method or to fulfill changing requirement from clients. This dynamic nature renders the project more complex and risky to manage, forcing project teams to busy communicate, coordinate and incorporate changes throughout the entire project period and avoid making further changes to improve the process.

- **Time and cost constraint for project completion**
  Completing project on time, within budget and quality requirement always pose great challenges to the project management teams which have to struggle between adversative conditions. Little time or budget is allocated to set up a formal and structured project feedback system, making effective learning impossible. CII found from their research that for construction firms without a quality system in place, feedback was based on an ad hoc basis, that is, whenever problems occurred [3].
• Industry Culture
Personnel in project management team prefer to carry out their tasks based upon past experiences, rather than following a formal or established approach. Project expertise becomes personal and is pervasively tacit. It is rarely acquired in an explicit form, and hardly ever shared among others in a structured way. Word of mouth is a common medium through which such expertise disseminates.

Besides the common barriers associated with the construction business characteristics, traditional organizational structures, management practices and organizational policies of the firms are also considered to be inadequate in a dynamic environment. They are found to be inflexible and hierarchical, failing to harness the ingenuity needed to solve unprecedented problems and grasp unpredictable opportunities.

The learning organization is an organization that is designed to enable learning and has an organizational structure with the capability to facilitate learning. Mirvis notes that the learning organization focuses on managing chaos and indeterminacy, flattening of hierarchies, decentralization, empowerment of people, teamwork and cross functional teams, network relationships, and the adoption of new technologies and new forms of leadership and mentoring [4].

PROPOSED IMPLEMENTATION PLAN TOWARDS LEARNING ORGANIZATION
To begin, the firms need to know the following: where they are as a learning organization; the objective level they are going to reach next; the barriers obstructing the firm from developing. They should analyze the reasons why such barriers exist, before planning the necessary changes to overcome these barriers, and develop a detail execution plan to lead the necessary changes. (Figure 1)

Table 1 – Learning Organization Levels (CII 2005)

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Learning Organization Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Organization</td>
</tr>
<tr>
<td>Leadership</td>
<td>Level 2</td>
</tr>
<tr>
<td>Processes and</td>
<td>Level 3</td>
</tr>
<tr>
<td>Infrastructure</td>
<td></td>
</tr>
<tr>
<td>Communication/</td>
<td>Level 4</td>
</tr>
<tr>
<td>Collaboration</td>
<td></td>
</tr>
<tr>
<td>Education</td>
<td>Level 5</td>
</tr>
<tr>
<td>Culture</td>
<td>Level 5</td>
</tr>
</tbody>
</table>

At the execution stage, people in all level in a firm play equally important role through the process. To promote organizational learning, there will be a few steps to galvanize people involving: first change initiative needs the top management’s realization of the importance and willing to provide necessary support to change; then leadership throughout the transformation process plays a vital role to push the change initiative; and finally the success to establish the new corporate culture of organizational learning relies on individuals at all level to support and implement.

CONCLUSIONS
International construction firms need to step back from their daily workflow of project tasks to think and reflect on what is going around them, notwithstanding the intense pressure of competition that requires them to complete projects in high quality, low cost and within time. They need to develop organizational learning capacities to ensure their business success and remain competitive in the rapidly changing international environment.

Based on the pilot survey and literature review, a conceptual framework for developing organizational learning for international construction firms is postulated. Further research and study are to be conducted to help the construction industry in general, and the construction firms in particular, to improve their competitiveness.

REFERENCES
A Modified Risk Management Process in PPP/PFI: The Real Option Element

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INTRODUCTION

Since the 1970s, privatization has been recognized as an alternative approach to solve the difficulty of funding large scale public projects due to resource constraints. In many ways, the private sector continues to play an important role in the development of infrastructure systems. Among the various channels and modes of participation, the Public Private Partnership (PPP) or the Private Finance Initiative (PFI) scheme is one of the most popular procurement methods adopted. With this procurement method, the government will not provide full or direct funding for the design and construction of public works (except for indirect guarantees, support and partial funding). Instead, through the PPP/PFI scheme, government has become a partner along with the private sector.

Since PPP/PFI projects are often perceived as more risky than those delivered using the conventional delivery modes (such as Design-Bid-Build and Design-and-Build), it is important for all project participants to manage the risks involved. In some ways, the existing risk management strategies are linked to the assumptions that underlie the traditional evaluation method using discounted cash flows (DCF). Put specifically, the appeal of alternative risk management strategies may be founded on the outcome of these discounted project values, which tend to ignore managerial flexibility in the project management process [1]. Real option is regarded as a better approach to evaluate risks or uncertainties in a project and lead to a more equitable risk sharing setting [2].

NEW RISK MANAGEMENT PROCESS INTEGRATED WITH REAL OPTION

Risk exists throughout all stages of a privately financed infrastructure project. Standard risk management process follows three typical steps: risk identification, risk measurement, and risk mitigation. Generally, risk mitigation approaches can be subdivided to include risk avoidance, risk reduction, risk shifting or transfer, and risk retention. Although these four strategies could help manage uncertainty and can effectively reduce the negative effects of uncertainty, they presume a certain degree of losses. As a matter of fact, the conventional approaches limit managers’ ability to recognize and exploit opportunities to increase project value. Accordingly, it is necessary to develop a risk management framework which considers the flexibility of management.

Figure 1 shows a new risk management process which incorporates the real option concept. The process includes risk identification and subsequent design of risk mitigation strategies. As confirmed by past literature, since real option is a better method to evaluate a project than NPV and it can enhance the value of a project, the process will adopt this as the first approach to manage risks. In this process, parties who are willing to assume total or partial risks will take an option approach to shape and manage the risks. For example, flexibility can be built into contracts by introducing special clauses which can be used to alter the timing and sequence of activities to achieve reduced risks. Naturally, however, this option approach could only shape and mitigate certain risks – there are often other residual risks that cannot be managed well by this way. It would then be helpful to realize the complementary nature of other conventional risk mitigation strategies in view of these residual risks that cannot be handled by the real option approach. As shown in the figure, these residual risks would filter through the option strategies and will be managed by traditional risk mitigation strategies: avoidance, reduction and transfer.

RISK CLASSIFICATION IN A PPP/PFI PROJECT

In Figure 1, the logical starting point of risk management is identification of potential risks before taking measures to mitigate the effects of these risks. Risks associated with privately financed projects can be identified by investigating risk events that appear throughout different stages of project development or by investigating the sources of these risks. This leads to various approaches of risk identification. The most common approach is the checklist approach. Li et al. [3] and Wang and Tiong [4] conducted insightful works on risk identification. Cheah and Liu [5] have also categorized risks that exist in a typical infrastructure project into two kinds: general risks and specific risks.

By consolidating the findings from past literature, Table 1 presents a checklist for some of the major risks that can be found throughout different stages of the project life cycle.

CONSTRUCTION
Table 1. A Checklist of Risks in Different Stages of Project Life Cycle

<table>
<thead>
<tr>
<th>Stages of Project Life Cycle</th>
<th>Risks/Uncertainties</th>
</tr>
</thead>
</table>
| Initial                     | • Delay in project approval and permits  
                              | • Design deficiency  
                              | • Availability of funds  
                              | • Increase in financing cost  
                              | • No experience of PPP/PFI projects  
                              | • Public opposition to project |
| Execution                   | • Land acquisition and compensation  
                              | • Geotechnical condition  
                              | • Construction cost overrun  
                              | • Construction time delay  
                              | • Quality of construction  
                              | • Variation in scope  
                              | • Default by concession company, contractor, or government  
                              | • Environmental damage  
                              | • Restriction on import equipment/materials  
                              | • Force majeure |
| Facility in Service         | • Demand risk  
                              | • Fluctuation of supply  
                              | • Government restriction on profit and tariff  
                              | • Operation cost overrun  
                              | • Inflation  
                              | • Foreign currency exchange rate  
                              | • Changes in law  
                              | • Payment failure by government  
                              | • Termination by government  
                              | • Government’s adverse action or inaction  
                              | • Labor risk  
                              | • Technology risk  
                              | • Condition of facility  
                              | • Force majeure |

According to the key functions of a project, its life cycle can be generally divided into three stages: initial stage, execution stage and the stage of facility-in-service. During the initial stage, the search for opportunities, feasibility studies, and planning and design of a project are conducted. The study of risks at this stage lays a solid foundation for a project. During the execution stage, key project functions would focus mainly on construction and installation of the facility. The physical construction process is often so complex that many risks and uncertainties are involved at this stage. Moreover, many activities are critical in terms of timing and sequences, and with typically large ticket items of capital expenditures to be incurred at this stage, it would be wise to introduce some flexible project management measures. These measures may include switching the sequence of certain construction activities, or seeking alternative sources of material supply. Finally, the stage of facility-in-service includes start-up, operation and maintenance of project facilities and also subsequent upgrading. For a PPP/PFI project, this is a key stage for the private sector to recoup their revenue and return. The nature of risks varies broadly within this stage (e.g. market risks, technology risks, currency exchange risks, inflation etc.) and the financial condition of the project is highly subjected to successful management of these risks. However, similar to the influence curve concept in project management (which purports that it would be most costly to make changes at the downstream of a project life cycle while having the least impact to influence operating performance), it may be too late to introduce flexible measures that have not been planned for – since actions taken at this stage would be subject to existing constraints due to pre-specified contract conditions at the earlier stage, components of the facilities that have been built and technologies that have already been adopted. Therefore, the degree of freedom and value of such ‘last-minute’ options are not likely to be high.

Table 2 matches the profile of certain options with the nature of the three stages of the project life cycle. Obviously, Table 2 only serves as a guide for general cases. A project sponsor who owns an expansion option may choose to expand the scale of the project at anytime so long as such action does not violate any contract specifications. The owner will take action according to his ongoing exposure to risks that change

Table 2. Profile of Options matching with Project Life Cycle

<table>
<thead>
<tr>
<th>Key functions</th>
<th>Initial</th>
<th>Execution</th>
<th>In Service</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nature</td>
<td>• Opportunity search</td>
<td>• Construction of facility</td>
<td>• Operation</td>
</tr>
<tr>
<td></td>
<td>• Feasibility studies</td>
<td>• Installation of services</td>
<td>• Maintenance</td>
</tr>
<tr>
<td></td>
<td>• Planning and design</td>
<td></td>
<td>• Upgrading</td>
</tr>
<tr>
<td>Sunk costs</td>
<td>Low</td>
<td>High</td>
<td>Varies</td>
</tr>
<tr>
<td>Degree of freedom to structure options</td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td>Most relevant types of options</td>
<td>• Deferment</td>
<td>• Switching</td>
<td>• Expansion/contraction</td>
</tr>
<tr>
<td></td>
<td>• Abandonment</td>
<td>• Expansion/contraction</td>
<td>• Switching</td>
</tr>
<tr>
<td></td>
<td>• Learning</td>
<td>• Abandonment</td>
<td>• Restructuring</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Learning</td>
<td>• Learning</td>
</tr>
<tr>
<td>Value of options</td>
<td>Highest</td>
<td>Moderate</td>
<td>Lowest</td>
</tr>
</tbody>
</table>
Impact of Political Risks on Public-Private Partnership (PPP) Opportunities in Asia

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Robert LK Tiong (clktiong@ntu.edu.sg)

ABSTRACT

Political risks have strong impact on opportunities in public-private partnerships (PPPs). The level of perceived political risks determines the costs in PPP projects. Political risks differ across Asian countries and PPP sectors.

This paper presents the results of a survey conducted among PPP practitioners from different backgrounds and countries on the impact of political risks on PPP opportunities in Asian countries and sectors.

INTRODUCTION

In Asian countries, the number of public-private partnership (PPPs) opportunities is increasing. Under PPPs, private sector entities provide public sector goods and services such as utilities, infrastructure, social services, and public real estate.

The success of PPPs depends on the successful identification, allocation, mitigation, and management of risks. Political risks are key risks in the public-private relationship.

Some political risks (A to D in Fig. 1) are insurable by public and private insurers while others (E and F) are not:
- A: Currency inconvertibility and transfer restriction (CI/TR)
- B: Expropriation
- C: Breach of contract
- D: Political violence
- E: Legal, regulatory, and bureaucratic risks
- F: Non-governmental action risks

The survey covers PPP opportunities in Bangladesh, Cambodia, China, India, Indonesia, Japan, Korea, Malaysia, Pakistan, Philippines, Singapore, Taiwan, Thailand, and Vietnam.

Figure 1: Influence diagram of political risks
PPP opportunities are possible in a number of sectors, and not all sectors are equally developed for PPPs. The survey covers following sectors:

- Utilities: power, water and waste water, and waste treatment and management
- Infrastructure: toll road, rail, sea port, and airport
- Social infrastructure and public real estate: mail, IT and telecommunications, education, health, leisure and sports, administration, and security

The respondents were asked to indicate their perception by using a seven grade scale from “extremely low” to “extremely high”.

The survey consists of three parts: the first one is on political risk assessment in PPP opportunities in Asian countries and sectors, the second one asks for perceptions on the likelihood and impact of the six political risk factors on financial decision criteria in specific projects, and the third asks for absolute values of these parameters.

**Ranking of political risks within countries**

The survey shows that within the Asian countries, the political risk factors vary in their perception. In Bangladesh for instance, currency inconvertibility and transfer restrictions are perceived as having the highest negative impact on possible PPP opportunities.

For most Asian countries, except Korea, Japan, and Singapore, legal, regulatory, and bureaucratic risks are perceived as having the strongest negative impact on PPP opportunities. These are followed by currency inconvertibility and transfer restrictions, breach of contract, then non-governmental action or outside risks, and expropriation. The least risky factor is political violence.

The picture differs significantly for the matured economies of Korea, Japan, and Singapore. They seem to bear little internal political risks and the worst political risk that may impact PPP opportunities in these 3 countries is induced by actions that are outside the control of the host governments. The second perceived riskiest factor is legal, regulatory and bureaucratic risks followed by breach of contract, political violence, and expropriation. The least critical factor in these economies is currency inconvertibility and transfer restrictions.

This comparison of survey results indicates that developers of PPPs in developing countries need to mitigate different risk profiles than that in matured economies. While developing and matured Asian economies have in common that breach of contract risk is higher than expropriation risk, it is the opposite for currency inconvertibility, political violence, legal and regulators risks, and non-governmental action risks. In fact, there is only a weak positive correlation between the two country sets of $r = 0.143$ and the level of significance of this correlation is less than 75%.

**Future PPP opportunities in Asian countries**

The survey responses indicate that in the near future, there will be less PPP opportunities than from 2010 to 2025. While in the developing Asian countries there will continuously be increasing opportunities from 2016 to 2025, the peak for PPP opportunities in the matured economies of Japan, Korea, and Singapore will be between 2010 and 2015.

Comparing the survey results on the future PPP opportunities by countries provides a more accurate picture.

While India is the single champion, Cambodia, Bangladesh, and Pakistan show least opportunities in PPPs throughout the years. Singapore ranks second best in 2007, but would drastically decline in the following years. Vietnam and Indonesia in contrary are most exceptional and consistently picking up in promising PPP opportunities. Malaysia is picking up but would be declining while Philippines is the reverse. Japan remains unchanged at rank seven. South Korea, China, and Thailand vary at a high rank level but show neither drastic up nor down developments. Taiwan starts from the third highest level and will at first pick up momentum but then decline and remain at a lower level.

**PPP SECTORS**

**Ranking of PPP sectors with respect to political risks**

In general, PPP opportunities in utilities are perceived to be riskiest, followed by infrastructure PPP opportunities. The survey suggests that social infrastructure and public real estate are least exposed to political risks.

**Ranking of political risks within sectors**

Except for the rail sector, legal, regulatory, and bureaucratic risks are the single worst risk category across all other sectors.

The risk profiles across the aggregated sectors correlate positively with each other. The legal, regulatory, and bureaucratic risks and non-governmental action risks are perceived to be the most critical ones. Currency inconvertibility and expropriation are perceived to be least risky. Political violence and breach of contract have mid-ranking across the aggregated sectors.

**Future PPP opportunities in sectors**

In general, the peak for PPP opportunities will be between 2010 and 2015 while in the near future, there are fewer opportunities than after 2016.

The mail, security, administration and education sectors promise the least PPP opportunities in the future. PPPs in power, water and waste water, and leisure and sports promise the greatest opportunities. Opportunities in rail and waste management and treatment are continuously increasing over the years, while IT/telecommunications, airports, and toll roads vary in the middle range. The health sector remains relatively unchanged and PPP opportunities in sea ports decline.

In aggregation, from 2007 to 2025, social infrastructure and public real estate promise the least PPP opportunities, while infrastructure is second and the most PPP opportunities are with the utilities.
## Project Specific Political Risk Quantification

With increasing political risks, investment appetite decreases. Also, with increasing political risks, the expected minimum internal rate of return (IRR), minimum required debt-service-coverage-ratio (DSCR), the risk margin on loans, and the insurance premiums increase. Investment appetite, loan spread, and insurance premium are chosen to best show the effect of political risks on the PPP project participants' objectives. The survey shows that there are also positive correlations between loan risk margins and minimum required DSCR, and positive correlations between investment appetite and minimum expected IRR.

The survey respondents were asked to provide their perception on the likelihood of occurrence and possible consequence of the six single political risk factors on these five financial decision criteria in specific projects. Also, respondents were asked to provide absolute values on project specific financial criteria as reference. The doctoral research on quantifying qualitative information on risks (QQIR) has been applied to quantify these perceptions. The project specific risk perceptions correlate with the general country and sector perceptions discussed before.

The numerical results (Table 1 and Table 2) fall within the actual market data, presented by HSBC.

### Table 1: Risk and return

<table>
<thead>
<tr>
<th>Direct survey response</th>
<th>QQIR validation</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of responses</td>
<td>true [%]</td>
</tr>
<tr>
<td>1. If the perceived political risks increase, the amount of investment decreases.</td>
<td>21</td>
</tr>
<tr>
<td>2. If the perceived political risks increase, the expected IRR will increase.</td>
<td>22</td>
</tr>
<tr>
<td>3. If the perceived political risks increase, the minimum annual DSCR will increase.</td>
<td>23</td>
</tr>
<tr>
<td>4. If the perceived political risks increase, the risk margin on a project loan will increase.</td>
<td>21</td>
</tr>
<tr>
<td>5. If the perceived political risks increase, the insurance premium will increase.</td>
<td>20</td>
</tr>
<tr>
<td>6. There is zero or a negative correlation between risk margin on loans and insurance premium.</td>
<td>21</td>
</tr>
<tr>
<td>7. With decreasing equity commitment, the risk margin on a project loan increases.</td>
<td>22</td>
</tr>
<tr>
<td>8. With decreasing equity commitment, the insurance premium increases.</td>
<td>21</td>
</tr>
<tr>
<td>9. With decreasing investment appetite, the minimum required DSCR increases.</td>
<td></td>
</tr>
<tr>
<td>10. There is a positive correlation between increasing investment appetite and expected IRR.</td>
<td>n.a.</td>
</tr>
<tr>
<td>11. There is a positive correlation between increasing minimum required DSCR and risk margin on loans.</td>
<td></td>
</tr>
</tbody>
</table>


<table>
<thead>
<tr>
<th>HSBC market data presentation</th>
<th>UK PFI</th>
<th>“Asia” PPP</th>
<th>QQIR results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leverage</td>
<td>&gt; 90%</td>
<td>60%–80%</td>
<td>72%</td>
</tr>
<tr>
<td>Bank Risk Margins</td>
<td>&lt; 100bp</td>
<td>&gt; 100bp</td>
<td>188bp</td>
</tr>
<tr>
<td>Base case equity risk premiums</td>
<td>12%–30%</td>
<td>10%–20%</td>
<td>14%</td>
</tr>
<tr>
<td>ADSCR</td>
<td>Min. 1.18–1.2</td>
<td>1.2–1.4</td>
<td>1.36</td>
</tr>
<tr>
<td>Tenor</td>
<td>&gt; 30 years</td>
<td>12–15 years – subject to market</td>
<td>n.a.</td>
</tr>
<tr>
<td>Maturity</td>
<td>1–2 years</td>
<td>2–5 years</td>
<td>n.a.</td>
</tr>
<tr>
<td>Insurance premium</td>
<td>n.a.</td>
<td>n.a.</td>
<td>115bp</td>
</tr>
</tbody>
</table>
CONCLUSIONS

The three data sets of survey findings are consistent and also correspond with current PPP market data. This survey has clearly shown that political risks have a significant impact on a country’s development. The higher the perceived political risks, the lower are chances for PPP opportunities in these countries and the costs for PPPs in such countries would increase as well. While for example Bangladesh, Cambodia, and Pakistan are perceived to be riskiest, they are also perceived to be least promising in PPP opportunities. A reason may be quoted from a survey respondent: “PPPs require stable legal and regulatory frameworks”. The survey also shows that in sectors, the risk-return perception may be different. While for instance the power sector is perceived to be among the riskiest project types, it also promises some of the highest future opportunities. In general, utility projects are perceived to be riskier than infrastructure projects or social infrastructure and public real estate projects. These also are perceived to be promising more opportunities.

ACKNOWLEDGEMENTS

Many thanks to all survey respondents for their time and effort in answering the questionnaire and responding to additional queries and evaluations.

The survey has been commissioned by James Neal, Ernst & Young, Partner, Global Head of Project Finance. The survey has been supported by Daniel Wagner, Senior Cofinancing Specialist (Guarantees), from the Office of Cofinancing Operations (OCO) at the Asian Development Bank (ADB), Bela Onken, Senior Project Manager, Team Project Finance, KfW-IPEX Bank, Prof Dr Hans Wilhelm Alfen, Chair of Construction Economics, Bauhaus University of Weimar (BUW) and Head of EU-Asia network for PPP enhancement, and Dr Andreas Wibowo, Ministry of Public Works, Indonesia.
Effect of Biofilm on Microfiltration Membrane for Ultrafiltration Quality Water Production

INTRODUCTION

Despite the many advantages of the membrane bioreactor (MBR), cost had its inhibited widespread inauguration. The most significant factors influencing the overall cost were power requirements, membrane costs, and membrane replacement frequency. This first factor was overcome with the emergence of submerged MBR which has lower power consumption. With the cost of membranes only a tenth of what it was ten years ago, the remaining factor would be the membrane replacement frequency and that would depend greatly on membrane fouling. Much work has also been dedicated to membrane fouling rate reducing. However, there is little information about the function of biofouling on treatment efficiency. The presence of biofouling could destroy membrane structural integrity and lead to system failure, causing irreversible membrane damage and increasing the operational and maintenance costs. However, the biofouling is thought to provide some protection to the membrane since such films are more selective than the membrane itself. This paper examines the possible effect of biofouling on microfiltration (MF) membrane in producing ultrafiltration (UF) quality water.

MATERIALS & METHODOLOGIES

Figure 1 shows the pilot scale setup of a submerged MBR. This system consists of an activated sludge bioreactor with a submerged MF membrane (0.9 μm) module. Detailed operating conditions of the experimental runs are summarized in Table 1.

During the cleaning phase, the membranes are first taken out of service. Then each ceramic membrane is jet-washed to remove the fouling layer. If only mechanical method is employed, the membrane is washed with a soft sponge. After that, the surface of the membrane is run under a tap for a few minutes before being placed back in service. If chemical cleaning is used, the membranes are soaked in 0.5% sodium hypochlorite (NaOCl) while placed in an ultrasonic bath for one hour to remove the internal fouling. Following that, the membranes are rinsed thoroughly with distilled water before being returned to service. In this study, the Standard Methods for Examination of Water and Wastewater (AWWA, 1998) was used for all sample analysis. Standard 0.45μm filters were used to separate the supernatant from the mixed liquor in the MBR. Results are used as comparisons with the 0.9 μm MF ceramic membrane filtered permeate.

RESULTS AND DISCUSSION

Figure 2 shows the soluble chemical oxygen demand (COD) profiles of MBR. COD values of the permeate was consistently low at less than 20 mg/L throughout the 823 days of operation. This would correspond to a high COD removal efficiency of more than 99%. However, this high rejection rate is non-typical for the MF membrane with a pore size of 0.9μm.

One of the major advantages of the submerged MBR is the complete removal of suspended solids (SS) due to the membrane filtration. SS in the permeate is non detectable as compared to 200 mg/L in the influent wastewater. However,

Table 1. Operating conditions of the submerged MBR

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duration (d)</td>
<td>823</td>
</tr>
<tr>
<td>Working volume (L)</td>
<td>20</td>
</tr>
<tr>
<td>HRT (hr)</td>
<td>8</td>
</tr>
<tr>
<td>MLSS (mg/L)</td>
<td>1200</td>
</tr>
<tr>
<td>Flux (L/m².hr)</td>
<td>26</td>
</tr>
<tr>
<td>VLR (kgCOD/m³.day)</td>
<td>3</td>
</tr>
<tr>
<td>Temperature (°C)</td>
<td>25</td>
</tr>
<tr>
<td>pH</td>
<td>6.5 - 8.0</td>
</tr>
</tbody>
</table>
the near zero value were due to be the precision of the method rather than reflect the true value. Therefore, the turbidity and colour tests were conducted for higher degree of accuracy. Turbidity readings were taken from random permeate samples throughout the experimental run and found to vary from 0.136 to 0.123. These values were half that of the local tapwater (0.28 NTU) used for comparison purposes.

Other turbidity values of clean water sources are also shown in Figure 3. Colour absorbance readings were taken from random permeate samples throughout the experimental run. The absorbance reading for the permeate (membrane filtered effluent) varied from 0.096 to 0.091 a.u./cm which was better than the MF effluent (0.15-0.3) as indicated in Table 2.

The biological performance results of the MBR system are exceptionally good and non-typical for the MF membrane with a pore size of 0.9 μm. Results are closer to performance of typical UF MBR system. A look at the SEM captures on the membrane surface depicted in Figure 4 shows tiny pores on the “base layer” of sizes smaller than the actual membrane pore sizes. This would explain the phenomenon of the MBR system achieving COD/TOC removal efficiency of more than 99%, which is the result of a “UF” biofilm layer. These tiny pores are exposed by “cracks” occurring on the surface layer as shown in Figure 5. Uplift arising from the air bubble rising along the membrane surface helps to induce shear stress which constantly “opened up” the cake layer.

<table>
<thead>
<tr>
<th>Wastewater types</th>
<th>Absorbance at 254nm (absorbance unit·cm⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary</td>
<td>0.5 – 0.8</td>
</tr>
<tr>
<td>Secondary</td>
<td>0.3 – 0.5</td>
</tr>
<tr>
<td>Nitrified Secondary</td>
<td>0.25 – 0.45</td>
</tr>
<tr>
<td>Filtered Secondary</td>
<td>0.02 – 0.4</td>
</tr>
<tr>
<td>Microfiltration effluent</td>
<td>0.15 – 0.3</td>
</tr>
<tr>
<td>Reverse osmosis effluent</td>
<td>0.05 – 0.2</td>
</tr>
</tbody>
</table>

During initial stages of membrane fouling, suspended microbial cells are transported with the fluid flow to solid surfaces (membrane pores and surface), where they may be absorbed. These absorbed cells grow, replicate and excrete extracellular polymer substances (EPS) which bind the cells together. The aggregates of cells, EPS and other particulate matter accumulated at the membrane surface are termed biofilms (Characklis & Marshall, 1990). It has been suggested that biofilms are composed of micro-colonies of microbial aggregates EPS and other particulate matter (Wolfaardt et al., 1994; Bishop & Rittmann, 1995; Costerton et al., 1995). This forms the basis of the sturdy base fouling layer. After the onset of the biofilms on the membrane surfaces, most foulants which are much larger than the membrane pore size (10-50μm) (Defrance et al., 2000; Huang et al., 2001) and are thus too big to enter the biofilm will form a fouling “cake layer”. Lewandowski and Beyenal (2004) also reported the presence of a base film and a surface film.
CONCLUSIONS

From this study, it is clear that the biofilm which acts as an external membrane or rather as a “biomembrane” which has a higher rejection rate than the MF ceramic membrane itself. In fact, the MF ceramic membrane in the MBR serves as a supporting structure to that layer of “biomembrane”. This is analogous to the principle of a hollow fibre asymmetric UF membrane. This explains the reason behind the production of UF quality water from MF ceramic membrane MBR.

REFERENCES


INTRODUCTION

Copper slag is the by-product from the pyrometallurgical production of copper. After extraction, copper slag is often used as an abrasive in shot blasting to prepare steel surfaces for painting. The widespread use of lead and other heavy metals in protective paints can result in an increase in heavy metals of the used slag, which is classified as a solid, hazardous waste. Dumping or disposal of such huge amount of slag may cause environmental and space problems. Long-term landfill disposal of copper slag in Singapore is not viable since substantial amount of them are produced each year and the potential leaching of heavy metals into ground water is of concern. Studies have been carried out to explore possible ways of utilizing waste copper slag. Its favorable physico-mechanical characteristics can be utilized to make products such as cement, fill, ballast, abrasive, aggregate, roofing granules, glass, tiles, etc. The recycling of waste copper slag as a fine aggregate substitute in concrete was investigated in this study.

MATERIALS AND METHODS

Copper slag characteristics
Table 1 presents the metal content of waste copper slag. The high percentage of iron (Fe) followed by Al, Ca, Cu and Mg is quite consistent with previous studies [1, 2].

Table 1. Metal compositions of waste copper slag

<table>
<thead>
<tr>
<th>Element</th>
<th>wt.%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ni</td>
<td>0.02</td>
</tr>
<tr>
<td>Co</td>
<td>0.02</td>
</tr>
<tr>
<td>Cd</td>
<td>0.03</td>
</tr>
<tr>
<td>Mo</td>
<td>0.05</td>
</tr>
<tr>
<td>Cr</td>
<td>0.07</td>
</tr>
<tr>
<td>Ba</td>
<td>0.09</td>
</tr>
<tr>
<td>Mn</td>
<td>0.10</td>
</tr>
<tr>
<td>Ni</td>
<td>0.12</td>
</tr>
<tr>
<td>Co</td>
<td>0.25</td>
</tr>
<tr>
<td>Cd</td>
<td>0.77</td>
</tr>
<tr>
<td>Mo</td>
<td>1.33</td>
</tr>
<tr>
<td>Cr</td>
<td>1.61</td>
</tr>
<tr>
<td>Ba</td>
<td>2.01</td>
</tr>
<tr>
<td>Mn</td>
<td>3.08</td>
</tr>
<tr>
<td>Fe</td>
<td>48.72</td>
</tr>
</tbody>
</table>
Concrete mix composition
In this study, the coarse aggregate: fine aggregate: cement ratio adopted was 3:2:1 and the water/cement ratios range from 0.4 to 0.6. The copper slag-fine aggregate (sand) replacement ratios were 0%, 20%, 40%, 60%, 80% and 100%. The detailed mix design is summarized in Table 2.

Leaching test
The leaching of heavy metals in both copper slag and cement mortar was determined using US-EPA Toxicity Characteristic Leaching Procedure (TCLP) method [3]. The more severe Modified Extraction Procedure (mEP) was also performed on the samples. In the mEP leaching test, the samples were initially treated with distilled water at a liquid to solid ratio of sixteen by weight. For the first six hours, the pH of the solution was maintained at 5.0 ± 0.2 by adding glacial acetic. At the end of the extraction period, water was added so that the total extraction solution is equal to 20 times the weight of the solids.

RESULTS AND DISCUSSION

Compressive strength
Figure 1 presents the strength of concrete versus the copper slag replacement ratio. From 0% to 80% replacement, the higher the copper slag replacement, the higher the concrete strength that can be obtained. This can be explained by considering the main factors that affect the strength of concrete i.e., the strength of individual constituents and the bonds between them.

Except for 91-day compressive strength curves, the optimum compressive strength was obtained at 80% copper slag replacement. Further increase in the copper slag replacement percentage would result in poor contact with cement paste which would lower the concrete strength. The higher 91-day strength curve for Mix 6 shows development of later strength in cement paste which provide better interlocking and bonds.

Porosity
The study on porosity was carried out using both the Brunauer-Emmett-Teller (BET) and Mercury intrusion porosimetry (MIP) methods.

Porosity of copper slag and sand using BET: From the BET tests, the porosity of sand was found to be $4.86 \times 10^{-3}$ cc/g, while that for waste copper slag was almost negligible as micropores were undetectable.

Porosity of cement mortar using MIP: The porosities of cement mortar samples from selected mix design are presented in Figure 2. It can be observed that the porosities of cement mortar for all mixes with copper slag addition were below that of the control mix (Mix 1). The angularity of aggregate is an important factor that affects the voids ratio i.e., increase in content of rounded shape or smooth texture aggregate percent would decrease the porosity. The glassy texture of copper slag explains the lower porosity for mixes with higher copper slag replacement ratio.

Figure 2 also shows that the pore size distribution curves of all mixes are similar at pore sizes larger than 1000nm (large capillary pores and macro pores); however, as the percentage of copper slag replacement in the concrete increases, the volumes of pore sizes smaller than 1000nm (small capillary pores and gel pores) decrease.

The decrease in porosity is consistent with the increase in compressive strength as discussed earlier. As pores smaller than 20nm has little effect on strength and for a given porosity, smaller pores as majority will lead to a higher strength [3], the lower strength in Mix 5 could be due to its higher percentage of pore sizes larger than 100nm.

<table>
<thead>
<tr>
<th>Mix no.</th>
<th>$\alpha_{\text{slag}}$ (% sand replaced by copper slag)</th>
<th>w/c ratio</th>
<th>Mix proportion for 1 m$^3$ of concrete mix (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.6</td>
<td>$M_w$</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>0.6</td>
<td>198</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>0.6</td>
<td>204</td>
</tr>
<tr>
<td>4</td>
<td>60</td>
<td>0.6</td>
<td>211</td>
</tr>
<tr>
<td>5</td>
<td>80</td>
<td>0.6</td>
<td>218</td>
</tr>
<tr>
<td>6</td>
<td>100</td>
<td>0.6</td>
<td>226</td>
</tr>
<tr>
<td>7</td>
<td>80</td>
<td>0.4</td>
<td>233</td>
</tr>
<tr>
<td>8</td>
<td>80</td>
<td>0.5</td>
<td>167</td>
</tr>
</tbody>
</table>

Table 2 Details of Mix Compositions

![Figure 1 Strength versus copper slag replacement ratio](image-url)
Although Mix 7 had the same copper slag replacement ratio as Mix 5 (80%), its lower water/cement ratio of 0.4 resulted in a lower porosity, giving rise to a higher compressive strength.

Leaching of heavy metals using TCLP
The TCLP results representing the leaching of metals from copper slag–cement mortars as well as the discharge criteria of the Singapore National Environment Agency (NEA) are shown in Table 3. Based on the NEA criteria, the TCLP results indicate that copper slag can only satisfy the criteria for landfill disposal, and that its S/S product can meet the criteria for discharge into water course.

Leaching of heavy metals using modified extraction procedure (mEP)
Comparing the mEP results with the copper slag TCLP results, the concentrations of heavy metals were much lower. The leaching of Ca interfered with the leaching of heavy metals. The encapsulation of cement matrix also helped to reduce the available surface area of copper slag, hence, reducing the heavy metals concentrations in the leachate.

Table 3. TCLP results for 28-day concrete

<table>
<thead>
<tr>
<th>Element</th>
<th>Copper slag</th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 5</th>
<th>Mix 6</th>
<th>Mix 7</th>
<th>Mix 8</th>
<th>NEA Criteria (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Landfill</td>
</tr>
<tr>
<td>Ca</td>
<td>50.6</td>
<td>2013</td>
<td>1915.3</td>
<td>1877.7</td>
<td>1806.9</td>
<td>2030.6</td>
<td>2094</td>
<td></td>
</tr>
<tr>
<td>Mg</td>
<td>4.536</td>
<td>63.264</td>
<td>64.815</td>
<td>76.24</td>
<td>65.764</td>
<td>236.519</td>
<td>92.653</td>
<td></td>
</tr>
<tr>
<td>Cd</td>
<td>0.068</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td></td>
</tr>
<tr>
<td>Cr</td>
<td>0.136</td>
<td>0.036</td>
<td>0.017</td>
<td>0.008</td>
<td>0.008</td>
<td>0.007</td>
<td>0.009</td>
<td></td>
</tr>
<tr>
<td>Cu</td>
<td>23.42</td>
<td>&lt;0.0004</td>
<td>&lt;0.0004</td>
<td>0.002</td>
<td>0.001</td>
<td>0.02</td>
<td>0.01</td>
<td></td>
</tr>
<tr>
<td>Mn</td>
<td>0.734</td>
<td>&lt;0.0004</td>
<td>&lt;0.0004</td>
<td>&lt;0.0004</td>
<td>&lt;0.0004</td>
<td>&lt;0.0004</td>
<td>&lt;0.0004</td>
<td></td>
</tr>
<tr>
<td>Ni</td>
<td>0.064</td>
<td>&lt;0.005</td>
<td>&lt;0.005</td>
<td>&lt;0.005</td>
<td>&lt;0.005</td>
<td>&lt;0.005</td>
<td>&lt;0.005</td>
<td></td>
</tr>
<tr>
<td>Pb</td>
<td>0.797</td>
<td>0.007</td>
<td>0.007</td>
<td>0.007</td>
<td>0.004</td>
<td>0.008</td>
<td>0.008</td>
<td></td>
</tr>
<tr>
<td>As</td>
<td>0.227</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>0.02</td>
<td>0.02</td>
<td></td>
</tr>
<tr>
<td>Zn</td>
<td>23.32</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td>&lt;0.001</td>
<td></td>
</tr>
<tr>
<td>Ba</td>
<td>0.981</td>
<td>0.488</td>
<td>0.35</td>
<td>0.183</td>
<td>0.119</td>
<td>0.198</td>
<td>0.264</td>
<td></td>
</tr>
<tr>
<td>Fe</td>
<td>85.77</td>
<td>&lt;0.002</td>
<td>&lt;0.002</td>
<td>&lt;0.002</td>
<td>&lt;0.002</td>
<td>&lt;0.002</td>
<td>&lt;0.002</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. TCLP and modified EP Results for 91-day concrete

<table>
<thead>
<tr>
<th>Element</th>
<th>Copper Slag (TCLP)</th>
<th>Mix 2 (TCLP)</th>
<th>Mix 2 (mEP)</th>
<th>Mix 5 (mEP)</th>
<th>NEA Criteria (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Landfill</td>
</tr>
<tr>
<td>Ca</td>
<td>5061.0</td>
<td>2672.2</td>
<td>5429.6</td>
<td>5251.8</td>
<td></td>
</tr>
<tr>
<td>Cr</td>
<td>0.136</td>
<td>0.016</td>
<td>0.017</td>
<td>0.016</td>
<td></td>
</tr>
<tr>
<td>Cu</td>
<td>23.42</td>
<td>0.002</td>
<td>2.540</td>
<td>10.73</td>
<td>100</td>
</tr>
<tr>
<td>Mn</td>
<td>0.734</td>
<td>&lt;0.001</td>
<td>0.732</td>
<td>0.855</td>
<td>50</td>
</tr>
<tr>
<td>Pb</td>
<td>0.797</td>
<td>&lt;0.01</td>
<td>0.008</td>
<td>0.015</td>
<td>5</td>
</tr>
<tr>
<td>As</td>
<td>0.227</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>5</td>
</tr>
<tr>
<td>Ba</td>
<td>0.981</td>
<td>&lt;0.0001</td>
<td>1.839</td>
<td>2.335</td>
<td>100</td>
</tr>
<tr>
<td>Fe</td>
<td>85.77</td>
<td>0.573</td>
<td>1.218</td>
<td>1.035</td>
<td>100</td>
</tr>
</tbody>
</table>
The mEP indicates that except for copper content, Mix 2 still satisfies the NEA standard for effluent discharged into water course. However, the mEP leachate of Mix 5 does not satisfy the standard for both copper and barium concentrations.

CONCLUSIONS

The results revealed that the compressive strength of the copper slag-concrete increased as copper slag replacement ratio increased. A better workability was observed when the copper slag replacement ratio was increased. TCLP showed that cement mortar was successful in encapsulating heavy metals. However, mEP indicated that leaching of copper and barium could be a potential problem if the concrete is subjected to severe leaching conditions.

The results also showed lower porosities in copper slag-concrete product compared to that of normal concrete. Hence, compared to sand, copper slag provides more favourable characteristics in improving the concrete performance.

REFERENCES


INTRODUCTION

To cater for future economic development, more land needs to be created through land reclamation. However, there has been a server shortage of fill materials for reclamation in recent years. On the other hand, there is a problem with the disposal of a huge amount of swage sludge produced every year from water reclamation plants. It would be an ideal solution if the sewage sludge can be used as fill for land reclamation. Studies have been carried out at NTU in the past years to develop economical methods for sewage sludge and other waste materials to be used for land reclamation without causing adverse environmental effect. Some of the findings obtained from the studies are summarized in a series of papers published (Chu et al. 2002; 2003; 2005a, b; 2006; Lim et al. 2004a, b; 2006, Goi et al. 2003). The major conclusion of the studies is that it is possible to convert sewage sludge into fill materials for land reclamation in a cost-effective way if a combined chemical and mechanical method is adopted to treat the fills made of sludge and other waste materials. By following this method, sludge will be mixed with other waste materials such as copper slag or marine clay and a small amount of binders such as cement or lime and the mixed materials will then be subjected to a pressure for consolidation. Laboratory studies have shown that sludge treated using this method has adequate geotechnical properties and meets the major environmental requirements (Chu et al. 2003; Lim et al. 2004b). This article discusses the practical considerations related to the use of this method in Singapore.

SUGGESTED METHODS

In the near future, there will only be two water reclamation plants and both will be near the sea. This makes it very convenient to use sludge for land reclamation. Two methods are suggested below.

At the moment, the sewage sludge has to be pressed before disposal. For the proposed methods, the sludge needs not to be pressed. This will result in a substantial saving. The raw sludge can be sent directly to a mixer to mix with other wastes such as copper slag or marine clay and binders such as cement. The optimum ratios for different wastes can be established by conducting laboratory tests. In the previous studies conducted at NTU using dewatered sludge, a ratio of 77% of sludge, 15% of copper slag and 8% of cement by weight, or 54% of sludge, 37.5% of marine clay and 8.5% of cement were found to be suitable.

In the first method, the mixed sludge can be pumped directly through pumping pipes into the designated reclamation area. A dike is required to contain the fill along the boundary of the reclamation area, as shown in Fig. 1. The dike does not have to be impervious as in the case of a land fill site. This is because the mixed sludge is low in permeability and the water inside the sludge mixture will not flow freely. This is particularly the case if the water inside the mixed sludge is to be dissipated using vacuum pressure via installed drainage layers.

Practical Considerations in the Use of Sewage Sludge for Land Reclamation

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Therefore, leaching of heavy metals or any other contaminates into the sea is unlikely. Horizontal drains will be placed on the seabed before the pumping of sludge. Prefabricated drain panels (Fig. 2) can be used as horizontal drainage layers. When the sludge fill has reached 3 to 4 m thick, another layer of horizontal drains can be installed. The drainage layers will be connected to vacuum pumps so the mixed sludge can be consolidated under vacuum pressure after it is disposed. After the sludge fill reaches the mean sea water level, a sand layer can be used to cap the sludge layer, as shown in Fig. 1. This method is suitable when the water table in the reclamation area is relatively shallow and the location is not far from the water reclamation plant. The advantage of this method is that there will be little environmental risks as the water from the sludge is centrally collected. The mixed sludge treated in this way will have geotechnical properties that are much better than the seabed marine clay. However, this method is only workable if the main purpose is to dispose sludge and other waste materials as the amount of sludge produced each year is not sufficient for land reclamation to be carried out in a speedy way or on a large scale.

The second method is to put the mixed sludge into woven geotextile bags, and then dump the bags from a barge on to seabed in the area where land reclamation will be carried out later. The size of the bags may be chosen in such a way that each bag will weigh between 30 to 50 kg. The thickness of the sludge bag layer can be controlled to be within a few meters according to the amount of sludge to be dumped. After sandfill or other fill material is placed subsequently as part of the normal land reclamation process, the bags will be covered by the fill material, as shown in Fig. 3. Under the fill surcharge, the sludge filled bags will consolidate through the drainage provided by the woven geotextile used for the bags. The bags will also sink into the marine clay under the fill. This method is suitable when the water depth of the reclamation area is deep. The advantage of this method is that no containment dikes are required. The disadvantage is that the dumping process is more labour intensive. However, this can be overcome by automating the bagging and dumping process. In this method, water from the sludge mixture is dissipated into the fill. However, this will not cause environmental concerns when the sludge is treated using binders as shown by the studies presented by Lim et al. (2004a,b). Furthermore, as only a thin layer of bagged sludge is deposited, the amount of water dissipation from the sludge will be small in any case.

**CONCLUSIONS**

Sludge and other waste materials can be converted into a sustainable source of supply for fill materials for land reclamation. The methods proposed may also be one of the most economical solutions to the sludge disposal problems. Therefore, these methods proposed should be evaluated together with other options in the sludge and waste management plans.

**REFERENCES**


INTRODUCTION

Anaerobic digestion is an attractive technology for the treatment of food waste because it reduces the volume of food waste, generates fuel biogas, mainly methane, and produces organic residue that can be used as soil conditioner or fertilizer.

In order to improve the efficiency of the process, the hybrid anaerobic solid-liquid (HASL) system was proposed to treat food waste in Singapore (Wang et al., 2005). This modified two-phase anaerobic digestion system includes an acidogenic reactor (Ra) to treat solid food waste and an upflow anaerobic sludge blanket (UASB) methanogenic reactor (Rm) to treat acidified leachate from Ra (Figure 1). Part of the effluent from Rm is used for the dilution of acid leachate from Ra to maintain optimal condition for methanogenesis, and the rest of the effluent from Rm is recycled into Ra to avoid addition of water for food waste hydrolysis.

Hydrolysis, the first step in anaerobic biodegradation of organic matter, ensures the breakdown of biopolymers of waste to water-soluble products, which can be easily consumed by microorganisms. Hydrolysis and liquefaction of organic waste often is rate-limiting step in anaerobic process (Shin et al., 2001). This study was to investigate leachate recirculation in the acidogenic reactor, thermal or frozen/thaw pre-treatment of food waste to enhance the hydrolysis stage and the overall process in the HASL system.

Enhancement of Food Waste Digestion in the Hybrid Anaerobic Solid-Liquid (HASL) System

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Figure 1. Schematic design of the HASL system
MATERIALS AND METHODS

Food waste was collected from a university canteen and shredded into particles with average size of 6.0 mm in a Robot-Coupe Shredder (CL50 Ultra, Hobart, France). Fresh food wastes were used in the control (C) and in experiment 1 (E1) with leachate recirculation in the acidogenic reactor. Thermally pre-treated at 150°C for 1 h food waste was used in experiment 2 (E2), while the food waste frozen for 24 h at -20°C, and then thawed for 12 h at 35°C, was used in experiment 3 (E3). All HASL systems were operated in a constant temperature room at 35±1°C for 12 days (Wang et al., 2006).

The leachate from Ra and the effluent from Rm were sampled for analysis daily. Chemical oxygen demand (COD) was determined by standard methods. Volatile fatty acids (VFA) were analyzed using HPLC (Perkin Elmer, Series 200, Norwalk, CT, USA). Biogas production was monitored by a wet gas meter (Ritter TG05, Germany), while gas composition was analyzed by a GC (HP5890A HACH, USA). The structure of the initial and thermally pre-treated food waste was observed by scanning electron microscopy (SEM) (Stereoscan 420, Leica Cambridge Instrument, UK).

RESULTS AND DISCUSSION

The COD and VFA productions in the leachate from Ra are shown in Figure 2.

The peaks of COD and VFA concentrations in all three experiments were higher and reached faster than that in the control, indicating intensification of hydrolysis and acidogenesis. There were big variations in COD and VFA concentrations between the control and experiments in the first 6 days (Figure 2). The highest COD concentrations were 16.9 g/l on 3rd day in C, 21.1 g/l on 2nd day in E1, 35.0 g/l on 1st day in E2 and 18.9 g/l on 1st day in E3. The highest VFA concentrations were 11.7 g/l on 3rd day in C, 20.9 g/l on 2nd day in E1, 22.7 g/l on 3rd day in E2 and 17.0 g/l on 1st day in E3.

During the first 6 days, the average ratio of VFA/COD was 61.4%, 71.5%, 69.1% and 68.6% in C, E1, E2 and E3, respectively. So the efficiency of acidogenesis process was higher in the experiments than that in the control. Thermal pre-treatment of food waste was the most effective method, then leachate recirculation in the acidogenic reactor and frozen/thaw pre-treatment of food waste were followed.

The rate of methane production in all experiments was much higher than that in the control (Figure 3).

The same volume of methane was produced in 12 days in the control, in 5 days in E1 with thermal pre-treatment of food waste, in 5 days in E2 with leachate recirculation in the acidogenic reactor, and in 6 days in E3 with frozen/thaw food waste. The time needed to produce the same quantity of methane was diminished by 58% using thermal pre-treatment of food waste or leachate recirculation in Ra, and by 50% using frozen/thaw pre-treatment of food waste.

The SEM images of fresh and thermally pre-treated food waste are shown in Figure 4. Thermal pre-treatment of food waste increased porosity and decreased bulk density, thickness and volume of the material. After heating at 150°C for 1 h, cavities appeared, and the structure of vegetable roots became loose (Figure 4b).

The HASL system was effective to treat food waste and to produce methane for energy recovery. The scaled-up HASL system has led to the establishing of a demonstration plant with a capacity of 3 tonnes of food waste per day (Figure 5).
The plant has been established in NTU campus and started operation in October 2006.

CONCLUSIONS

Thermal pre-treatment of food waste and leachate recirculation in the acidogenic reactor were more effective methods to facilitate the hydrolytic and acidogenic processes. They enhanced food waste digestion in the HASL system in terms of higher and faster COD, VFA and methane production. Application of these pre-treatment methods diminished the time needed to produce the same quantity of methane by more than 50%.

REFERENCES


INTRODUCTION

In the study of aerobic granulation, acetate has been commonly used as carbon source, and a high calcium content has often been reported in acetate-fed aerobic granules even though the calcium concentration in the nutrient medium was very low (Qin et al., 2004; Wang et al., 2005). So far, little is known about the reason behind the excessive accumulation, chemical form and spatial distribution of calcium ion in acetate-fed aerobic granules. This study aimed to offer in-depth insights into the mechanism of calcium accumulation in acetate-fed aerobic granule.

MATERIALS AND METHODS

Cultivation of aerobic granules
Mature acetate-fed aerobic granules were harvested after two months of cultivation in a column sequencing batch reactor (SBR) with a diameter of 5 cm and a working height of 100
Elemental analysis of aerobic granules
The elemental composition (Ca, Mg, P, Fe, Al) of acetate-fed aerobic granule was determined by Inductively Coupled Plasma Emission Spectrometer (ICP, PerkinElmer Optima 2000). To analyze the amount of carbonate ion in acetate-fed aerobic granules, 3 ml of 1 M hydrochloric acid solution was added to 50 ml of 2 g SS L⁻¹ granules. The carbon dioxide gas produced was online measured by a carbon dioxide sensor (Columbus Instruments Micro-Oxymax), while change in inorganic carbon in the liquid phase was determined by Total Organic Carbon Analyzer (TOC, Shimadzu TOC-Vcsh) before and after the experiment. Thus, the content of carbonate in acetate-fed aerobic granules can be calculated.

Calcium mapping by EDX
The calcium distribution in acetate-fed aerobic granule was investigated using a scanning electron microscope (SEM, JSM 6360, JEOL, Tokyo Japan) as well as Energy Dispersive X-ray spectroscopy (EDX) analysis. The carbonate localization was determined by a chemical titration method, and evolution of gas bubbles was then visualized by Image Analysis technique (IA, Quantimet 500, Leica Cambridge Instruments).

RESULTS
Chemical form of calcium in acetate-fed aerobic granules
Fig. 1 shows the major inorganic components of acetate-fed aerobic granule. It can be seen that both Ca²⁺ and CO₃²⁻ are dominant ions over the other inorganic components, such as Mg, P, Fe and Al, which are indeed negligible. As shown in Fig. 1, the molar ratio of granule calcium to carbonate was estimated as 1:1.16, indicating that most calcium ions in aerobic granule exist in the form of calcium carbonate.

Calcium distribution in acetate-fed aerobic granules
Fresh aerobic granules with a SOUR of 64 mg O₂ g⁻¹ VS h⁻¹; SVI of 52 ml g⁻¹ and a mean diameter of 1.4 mm were sectioned for SEM-EDX analysis (Fig. 2). Figs. 2a and 2b show that calcium is accumulated in the core part of aerobic granule, while the granule shell is nearly calcium-free. The IA analysis further reveals white deposits localized at a depth of 300 µm from the granule surface (Fig. 2c). After hydrochloric acid was added to the zone of white deposits (Fig. 2c), gas bubbles were immediately generated (Fig. 2d). The gas phase analysis shown in Fig. 1 confirms that the bubbles generated were carbon dioxide (Fig. 2d). These observations provide further visualized evidence that calcium mainly exists in the form of CaCO₃ in acetate-fed aerobic granules, which is in good agreement with the stoichiometric analysis (Fig. 1).

DISCUSSION
Both microscopic observation and chemical analysis show that the accumulated calcium is present in the form of CaCO₃, and is found in the central part of acetate-fed aerobic granule (Figs. 1 and 2). Compared to acetate-fed aerobic granules, Fig. 3 shows that the calcium and ash contents were very low in aerobic granules grown on ethanol as sole carbon source. This is mainly due to the fact that alkalinity in the form of hydroxide ion was produced during the biological oxidation of acetate (Eq. 1), while no hydroxide ion can be generated in the oxidation of ethanol (Eq. 2). In this case, calcium would not be accumulated in ethanol-fed aerobic granules (Fig. 3).
\[ \text{CH}_3\text{COO}^- + 2\text{O}_2 + \text{H}^+ \rightarrow 2\text{CO}_2 + \text{H}_2\text{O} \]  \hspace{1cm} (1) \\
\[ \text{CH}_3\text{CH}_2\text{OH} + 3\text{O}_2 \rightarrow 2\text{CO}_2 + 3\text{H}_2\text{O} \]  \hspace{1cm} (2)

**CONCLUSIONS**

This study shows that the majority of calcium was present in the central part of acetate-fed aerobic granule and the granule shell part was nearly calcium free. Moreover, the calcium ions accumulated in acetate-fed aerobic granule mainly existed in the form of calcium carbonate (CaCO\(_3\)).

**REFERENCES**


Expansive Behaviour of Clay Mixtures

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INTRODUCTION

Expansive soil is soil material that has a potential for both shrinking and swelling under moisture variations (Nelson, 1992). Expansive soils are found in many parts of the world, particularly in semi-arid areas. In a review from Chen (1988), expansive soils are detected in Australia, Canada, China, Israel, Jordan, Saudi Arabia, India, Sudan, South Africa, Spain and the United States. In addition, expansive soils are also found in several areas in West Java, Indonesia, i.e. in the northern part and central part of West Java (Wibawa, 2003).

Expansive soils show various responses to moisture variations. First, expansive soils exhibit significant volume change when there is moisture variation: it shrinks due to decrease in moisture, and it swells when it absorbs water. For example, the volume of expansive soils can increase up to eight times of the original volume when it absorbs water from the surroundings (Nelson, 1992). The volume change of soil during moisture variations is mainly due to the presence of expanding clay mineral—montmorillonite. Second, expansive soil exerts swelling pressure to the surrounding during moisture increase. As a result, structures built on expansive soil experience extensive damage due to the large uplift forces caused by the swelling effect.

Swell pressure is a pressure which is required to return the specimen back to its original state, i.e. void ratio or height before swelling (ASTM D-4546). There are various methods to measure swell pressure of expansive soils (Nelson, 1992). First, it is called double oedometer method, a consolidation-swell test under a small surcharge pressure and a consolidation test, performed in the conventional manner but at natural moisture content to measure swell pressure. Various loading conditions and final pore-water pressures are allowed in this method. Second, an improved version of the double oedometer method, simple oedometer, allows a single sample loaded to overburden, then unloaded to constant seating load, immersed and allowed to swell, followed by usual consolidation procedure in order to measure swell pressure. Lastly, it is called a constant volume test to measure swell pressure which can make a correction for sample disturbance and apparatus deflection (Fredlund et al, 1990).

The swelling behaviour of clay mixtures due to moisture variations is still unclear. The objective of the paper is to investigate the effect of molding water content and percentages of bentonite on swell pressure of clay mixtures using the simple oedometer method.

LABORATORY EXPERIMENTS

Kaolin is a group of clay minerals dominated by kaolinite consisting of repeating layers of a tetrahedral sheet and an octahedral sheet that are held together by strong hydrogen bond. Kaolin is derived primarily from the alteration of feldspar and micas. Kaolin is also an inert mineral due to its lower specific surface and absorbed water capacity when compared to other clay minerals (Holtz and Kovacs, 1982). Table 1 shows the basic soil properties tested for both kaolin and Na-bentonite.

<table>
<thead>
<tr>
<th>Clay Minerals</th>
<th>Plasticity Index (%)</th>
<th>Specific Gravity</th>
<th>OMC (%)</th>
<th>Max. Dry Density (Mg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolin</td>
<td>21</td>
<td>2.64</td>
<td>25</td>
<td>1.368</td>
</tr>
<tr>
<td>Na-Bentonite</td>
<td>320</td>
<td>2.71</td>
<td>39</td>
<td>1.106</td>
</tr>
</tbody>
</table>

Simple oedometer method, from ASTM test method D-4546 method A, was used in the experiments. The purpose of the method was to measure the free swell, percent heave and swelling pressure using compacted specimens of bentonite and kaolin mixtures. The modified oedometer cell was used for the setting of the specimen in the simple oedometer method shown in Figure 1. Figure 1 indicates various detail components of the apparatus: cap and porous stone, drainage valve, cutter, ring, and pressure transducer valve.

EXPERIMENT RESULTS

Figure 2 shows swell of the specimen of 90% kaolin and 10% bentonite with time at a molding water content of 28%. Swell is the absolute heave of the specimen. The swell of the compacted specimen shows a very small gradient initially until around 20 minutes, and then the gradient increases significantly. This part of the curve is called primary swell: an arbitrary short-term swell usually characterized as being completed at the intersection of the tangent of reverse curvature to the curve of a logarithm of time plot with the
tangent to the straight line portion representing long-term or secondary swell (ASTM D-4546). When the gradient starts to decline, this is the end of the primary swell stage (~500 min.), and then a secondary swell stage occurs until the specimen stabilizes. Secondary swell is an arbitrary long-term swell usually characterized as the linear portion of a logarithm of time plot following completion of short-term or primary swell (ASTM D-4546).

Figure 2. Swell of the specimen of 90% kaolin and 10% bentonite at \( w = 28\% \)

Figure 3 illustrates typical pore-water pressure of the specimen of 90% kaolin and 10% bentonite at a molding water content of 28%. The pore-water pressure of the compacted specimen has almost zero gradient initially until around 20 minutes, and then the gradient increases significantly towards negative pressure. This part is related to the primary swell stage. When the gradient starts to decline, this is related to the end of the primary swell stage, and then a secondary swell stage, as shown by the positive gradient occurs.

Figure 4 shows a typical swell consolidation test using the simple oedometer method (ASTM D 4546-96) for the specimen of 90% kaolin and 10% bentonite at a molding water content of 28%. Figure 5 indicates that the swell pressure of the compacted clay mixture specimen is 45 kPa. The swell pressure has a relatively small value as the percentage of bentonite is quite small (10%).

Figure 5 shows the relationship between swell pressure and percentages of bentonite in the clay mixture for a water content of 28%. Figure 5 indicates that an increase in the percentage of bentonite in the clay mixtures results in a higher swell pressure value. What is the role of bentonite? Bentonite which is largely composed of a group known as montmorillonite consists of repeating layers of two tetrahedral sheets and an octahedral sheet that are held together by van der Waals’ force. Bentonite is generated from an in-situ alteration of volcanic ashes; it has low permeability and high plasticity and swell capacity when wet due to its water absorption capacity. Water absorption capacity is one of the most distinct characteristic qualities of bentonite, which depends on their content of montmorillonite, fineness of grain and cation occupancy (Holtz and Kovacs, 1982). As a result, the clay mixture exerts a higher swell pressure with the higher percentages of bentonite in the clay mixture. This is probably because the higher amount of bentonite in the clay mixture increases its ability to absorb water, thus resulting in swell pressure.

Figure 6 illustrates the relationship between swell pressure and molding water content of the clay mixture specimens for 90% of kaolin and 10% of bentonite. Figure 6 indicates that swell pressure decreases with an increase in the molding water content which is consistent with Chen (1988). This is probably because the higher the molding water content of the clay mixture, the lower water adsorption capacity of the mixture. In short, swell pressure of the clay mixture will decrease with the molding water content.

ANALYSIS

Figure 5. The effect of percentages of bentonite on the swell pressure
CONCLUSIONS

Within the scope of the experiment, the data trends reveal the following:

i) Swell pressure increases along with an increase in the percentages of bentonite in the compacted clay mixtures.

ii) Swell pressure decreases along with an increase in molding water content in the compacted clay mixtures.

REFERENCES


INTRODUCTION

The total stress or alpha ($\alpha$) method is commonly used to estimate the undrained shaft resistance of bored piles. Most studies have shown that the empirical adhesion factor $\alpha$ is correlated to the undrained shaft resistance and the undrained shear strength $s_u$. For example, from their regression analysis of 127 field load test data in which the $s_u$ is converted to a consistent test type consolidated-isotropically undrained compression (CIUC), Chen and Kulhawy (1994) proposed the following:

$$\alpha = 0.31 + 0.17\frac{s_u (CIUC)}{p_a}$$

in which $p_a =$ atmospheric pressure. The inter-relationships between the different measured strength values were determined from correlations developed by Kulhawy and Mayne (1990). In this study, the database from Chen and Kulhawy (1994) has been reanalyzed using a Bayesian neural network. In an extension to the original Bayesian neural network algorithm developed by Mackay (1991), Chua and Goh (2003) incorporated the use of the genetic algorithms (GA) search technique and a higher-order search algorithm, the Levenberg-Marquardt algorithm. A distinct advantage of the Bayesian inference procedure is that every neural network prediction is associated with an error bar. These error bars are the standard deviations for the predictions based on the data distribution and inherent noise.

RESULTS

The undrained side resistance database from Chen and Kulhawy (1994) has been reanalyzed using the hybrid Bayesian neural network. The database for the neural network analysis was compiled from 127 field load tests on bored piles in a variety of cohesive soil profiles. In this study, the undrained shear strength values were the $s_u$ (CIUC) values or their equivalents as outlined previously. The mean effective overburden stress $\sigma_{vm}$ are in the range of 11 to 343 kPa. All the patterns had $s_u (CIUC)/\sigma_{vm}$ ratios (denoted as the undrained strength ratio USR) in the range of 0.49 to 6.9, except for two patterns with $s_u (CIUC)/\sigma_{vm} = 16.2$. The dataset
was separated randomly into a training set of 85 patterns and a testing set of 42 patterns and are described in Goh et al. (2005). The neural network structure used in the analyses consisted of two input neurons representing $\sigma'_{vm}$ and $s_u$ (CIUC), four hidden neurons, and an output neuron representing $\alpha_{CIUC}$.

The mean squared error for the predictions based on the neural network model is summarized in Table 1 alongside the standard deviation and coefficient of correlation $R^2$ of the load test versus predicted values of $\alpha_{CIUC}$. The high coefficient of correlation implies that the neural network model is reasonably accurate in its predictions.

![Table 1. Performance of neural network model for predicting $\alpha_{CIUC}$.](image)

Some parametric studies were carried out using the trained neural network model to examine further the adhesion factor behaviour. Some typical results are shown in Fig. 1 which show the neural network predictions of $\alpha_{CIUC}$ for $\sigma'_{vm}$ in the range of 25 to 200 kPa, and $s_u/\sigma'_{vm}$ in the range of 0.75 – 3, alongside the load test results that were close to the corresponding $s_u/\sigma'_{vm}$ ratios. The results show that the neural network model gives logical and consistent trends with the load test results. Both the load test and neural network results suggest that the in situ effective overburden stress $\sigma'_{vm}$ directly or indirectly has an influence on the value for bored piles. The plots indicate that the relationship between $s_u$, $\sigma'_{vm}$ and $\alpha_{CIUC}$ is rather complicated. The general trend was for the $\alpha_{CIUC}$ value to decrease with increasing $\sigma'_{vm}$ and with increasing $s_u/\sigma'_{vm}$. As the undrained strength ratio $s_u/\sigma'_{vm}$ is related to overconsolidation ratio OCR, the results appear to indicate that there is a correlation between $\alpha_{CIUC}$ and OCR. Full results are given in Goh et al. (2005).

**SUMMARY**

In general, the neural network predictions of the undrained shaft resistance alpha factor for bored piles agree well with the load test results. Both the load test results and the parametric studies using the trained neural network suggest that the in situ effective overburden stress $\sigma'_{vm}$ and OCR directly or indirectly have an influence on the value $\alpha_{CIUC}$ for bored piles. More data is required to confirm these findings.

**REFERENCES**


Wave Propagation Modeling of PZT Sensing Region for Structural Health Monitoring

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INTRODUCTION

Piezoceramic transducers (PZT) have been widely used for structural health monitoring (SHM) in a great variety of engineering structures due to their unique ability of generating high frequency elastic waves. The high frequency waves ensure the PZT’s sensitivity to structural damage, but they also limit the PZT’s sensing region. Up to date, there have been no systematic guidelines or established models to identify the sensing region of PZTs. Before the broad use of PZTs for SHM is possible, fundamental research on determining the transducers’ sensing region is necessary. Among a variety of issues in relation to the PZT’s sensing region, this article focuses on the material and structural damping, which has enormous influence on the energy dissipation of high frequency waves.

PROPAGATION OF PZT GENERATED WAVES

Due to the high frequency range employed, the Timoshenko beam theory, including the effect of rotary inertia and transverse shear deformation, is used to model the beam structure. The equations of motion are expressed in terms of transverse displacement and angle of rotation \( \psi \) as

\[
GAK \frac{\partial^2 \psi}{\partial x^2} + \frac{\partial}{\partial x} \left( \rho \frac{\partial y}{\partial t} \right) + \rho A \frac{\partial^2 y}{\partial t^2} = 0
\]

where \( \omega = \pi^2 / 12 \) and denotes the shear correction factor; \( \rho \) is the mass density; \( A \) is the cross sectional area; \( f \) is the inertia; \( E \) is Young’s modulus; and \( G \) is the shear modulus.

The spectral solution can be expressed as

\[
y(x,t) = \sum_{n=1}^{\infty} y_n(x) e^{i\omega_n t}
\]

Substituting Eq. (2) into Eq. (1) yields,

\[
EIK \frac{\partial^4 y}{\partial x^4} - \rho A \frac{\partial^2 y}{\partial t^2} + \rho \frac{\partial^2}{\partial t^2} \frac{\partial}{\partial x} \psi - \rho A \frac{\partial^2}{\partial t^2} \frac{\partial y}{\partial t} = 0
\]

The solution to this dispersion equation is

\[
k^2 = \frac{1}{2} \left( \frac{1}{C^2_1} \right) \pm \sqrt{\frac{C^2_2}{C^2_1}} \omega^2
\]

The amplitudes of waves can be expressed in terms of a series of matrices. For the beam structure shown in Figure 1, positive-going and negative-going wave vectors at points A, B and C are denoted as \( a^+ \), \( b^- \), \( c^- \), \( d^- \), \( g^- \) and \( a^- \), \( b^- \), \( c^- \), \( d^- \), respectively. The beam is assumed to be infinitely long to the right hand side of point C along the x-axis.

The wave approach can provide a concise and systematic vibration analysis of a build-up structure [1]. The displacement amplitudes of waves can be expressed in terms of a series of matrices. For the beam structure shown in Figure 1, positive-going and negative-going wave vectors at points A, B and C are denoted as \( a^+, b^-, c^- \) and \( a^-, b^-, c^-, d^- \), respectively. The beam is assumed to be infinitely long to the right hand side of point C along the x-axis. According to the coupling relation between the beam and PZT actuators, the force applied to the structure at point C can be derived as

\[
\vec{F} = K_A \alpha \left( e_{s} - e_{g} \right)
\]

The bending moment is

\[
\vec{M} = (h_s + h_g) \vec{F}
\]

where \( K_A = \frac{Y_{11} b h}{a \rho} \); \( e_{s} = \frac{h_0 \partial \psi}{\partial x} \); \( e_{g} = d_31 \vec{V} / h_p \); \( a \), \( b \), \( h \) and \( h_p \) are the length, width and thickness of the PZT patch, respectively; \( d_31 \) and \( Y_{11} \) are PZT material constants; and \( \vec{V} \) is the amplitude of the voltage applied to the PZT patches. After a series of substitutions and calculations, the amplitudes of the waves propagating in the beam at the right hand side of point C can be expressed in the form of external moments as

\[
g^* = \vec{T} \cdot \vec{M}
\]
where
\[
\mathbf{T} = \left( \begin{array}{c} t \end{array} \right) = \left( \begin{array}{c} t_i(x) \end{array} \right) \mathbf{R} \left( \begin{array}{c} t(x) \end{array} \right)
\]

\[
f(x) = \left[ \begin{array}{cc} e^{-a_1 x} & 0 \\
0 & e^{-a_2 x} \end{array} \right]
\]

\[
\mathbf{R}_{\text{free}} = \left[ \begin{array}{ccc} P_i & Q_i & 0 \\
0 & Q_i & 0 \\
0 & 0 & 0 \end{array} \right]
\]

\[
\mathbf{L} = \left[ \begin{array}{cc} 0 & 1 \\
0 & 1 \end{array} \right]
\]

\[
\mathcal{M} = \frac{d_{ij} \hat{V}}{2 \left( -k_i P_i + k_j Q_j \right)} - \frac{1}{Y_{02}^2 h_i h_j (h_i + h_j)}
\]

Using the correspondence principle of linear viscoelasticity, we can readily extend the elastic solution to the corresponding viscoelastic solution simply by replacing \( E \) by \( E' \), and \( G \) by \( G' \) [2]. Denoting Eq. (3) as \( f(k, E, G) = 0 \), the viscoelastic dispersion relation becomes \( f(k', E', G') = 0 \).

By carrying out a Taylor expansion, we obtain
\[
f(k', E', G') = f(k, E, G) + E' \frac{\partial f}{\partial E} + G' \frac{\partial f}{\partial G} + k' \frac{\partial f}{\partial k}
\]

where \( E'' = E' + E'' \), \( G' = G' + G'' \) and \( k'' = k'' \).

Therefore
\[
k'' = - \frac{E'' \frac{\partial f}{\partial E} + G'' \frac{\partial f}{\partial G}}{\frac{\partial f}{\partial k}}
\]

The viscoelastic mechanical strain is
\[
\hat{\varepsilon}_{\text{mech}} = \frac{h_i}{2} \left( -k_i P_i e^{-a_1 x} + k_j Q_j e^{-a_2 x} \right) \hat{M}
\]

According to constitutive equations of piezoelectric materials, the electric displacement of the PZT sensor is given by
\[
D_z = d_{33} Y_{11} k' \hat{\varepsilon}_{\text{mech}}
\]

The voltage \( V_{out} \) generated across the sensor electrodes is related to the capacitance of the sensor \( C_p \) as
\[
V_{out} = \frac{D_z a_\rho b_p}{C_p}
\]

**EXPERIMENT AND TEST**

Figure 2 schematically shows the overall experimental setup and the test specimen. The two PZT actuators are counter connected to the input slot of an HP4192A Analyzer. A sinusoidal sweep voltage with amplitude of 1 volt is applied to the two actuators to excite the system with a pure bending vibration over various frequency ranges. Two PZT patches serve as actuators and the other five as sensors. The PZT sensors are individually connected to different channels of the switch box which is connected to the output slot of the analyzer to record the signals. A box filled with sand is used to dampen the waves at one end of the aluminum beam specimen to achieve negligible wave reflection at the boundary, thus simulating a semi-infinite beam condition.

**RESULTS AND DISCUSSION**

Using the developed theoretical model, the sensing voltage of each sensor along the beam structure can be calculated. The output voltages calculated simulate the signals detected by the five sensors which are 0.2m, 0.5m, 1.5m, 2.1m and 2.9m away from the actuators, respectively. Peaks appear when the excitation frequencies approach the beam’s natural frequencies due to resonance.
The theoretical and experimental peak values of each sensor at an excitation frequency of 155 kHz are shown in Figures 3 and 4, respectively. Good agreement between the theoretical predictions and experimental results can be observed.

CONCLUSIONS

Based on the wave propagation theory, this study developed a method to determine the sensing region of PZT sensors for their applications in SHM. Results showed that the sensing region of PZT sensors is dependent on the excitation frequency. At higher frequencies, the sensing region is smaller due to the significant effect of material and structural damping. The sensing region also highly depends on the experimental conditions as well as the measurement precision of the equipment. Under this experimental situation, when the output voltage of PZT sensors is less than 0.01 Volt, it is difficult for the Analyzer to steadily record this small voltage. Therefore, it is concluded that the valid sensing region of the PZT sensors is about 2-2.5m in aluminium (see Figure 5).

REFERENCES


FINITE ELEMENT ANALYSIS OF REINFORCED CONCRETE SLABS UNDER FIRE CONDITIONS

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INTRODUCTION

In this paper, a non-linear shell finite element developed for modelling reinforced concrete (RC) slabs in fire condition is described. Both geometric and material nonlinearities were taken into account in the formulations. The geometric nonlinearity was implemented using co-rotational approach due to its efficiency in small strain large rotation problems and good convergence properties. Degenerate thick shell elements employing a layered discretization through thickness were adopted for the RC non-linear analysis. The finite element procedure was also used to model existing tests to verify the capabilities of the element.

DEGENERATE SHELL ELEMENT

Concrete slabs are modeled as an assemblage of quadratic degenerated thick shell elements. The elements can be 8-noded Serendipity, 9-noded Lagrangian or Heterosis elements. Five degrees of freedom are specified at each node corresponding to three translations and two rotations. The layered model as shown in Fig. 1 is used to discretize and integrate through the slab thickness. The layers can represent concrete or distributed reinforcing steel. Each layer can be subjected to different but uniform temperature. The initial material properties of each layer may be different, and the stress-strain relationships may change independently for each layer.

GEOMETRICAL NONLINEARITY MODELLING AND CO-ROTATIONAL APPROACH

The Co-rotational (CR) approach was used to model the geometric nonlinearity in this study. Compared to the total
Lagrangian and updated Lagrangian formulations, which are widely used to describe large deformation problems, the CR-approach is less well-known. However, it offers exceptional benefits for structural problems with rotational degrees of freedom, particularly when accounting for arbitrarily large rigid body rotations. In the CR-approach, a local coordinate system is attached to each element, and the system continuously translates and rotates with the element as the deformations proceed. The stress and strain fields are referred to the latest configuration. Moreover, this approach can be developed and implemented quite independently of the specific element formulation. It can act as a ‘harness’ around geometrically linear elements, rendering them immediately applicable to the modelling of geometric nonlinearities.

In the CR-approach, it will be assumed that the co-ordinate axes for the element’s local co-rotating frame \( e_1 \) \(-e_3 \) are given beforehand, as well as element’s nodal displacement vectors in global co-ordinates, \( d \). We can (at the element level) find the local nodal displacement vector, \( d_l \), via:

\[
d_l = f(d, e_1, e_3, e_5)
\]

On differentiating equation (1) and obtaining the transformation matrix \( T \):

\[
\delta d_l = T \delta d
\]

Based on the equivalence of virtual work in the local and global systems, the following relationship between global and local force vector is obtained:

\[
p = T^T p_l
\]

where element local internal force vector, \( p_l \) is given by the conventional relationship:

\[
p_l = \int B_l^T \sigma dV = K_l d_l
\]

It should be noted that \( B_l \) is the conventional strain/displacement matrix; \( \sigma \) is the current stress vector satisfying the yield condition; \( K_l \) is the element local tangent stiffness matrix allowing for the possibility of “local non-linearity”.

Thus, the global tangent stiffness matrix, \( K \) can be obtained by differentiating equation (3):

\[
\delta p = T^T \delta \sigma + \delta T^T p_l = T^T \delta \sigma + K_{\sigma} \delta d
\]

\[
= T^T K_{\sigma} \delta d + K_{\sigma} \delta d = K \delta d
\]

where \( K_{\sigma} \) is the initial stress stiffness matrix.

CONSTITUTIVE MODELLING AND PROPERTIES OF MATERIAL

In this study, several of the more complex features of structural behaviour of reinforced concrete slabs in fire conditions, such as dimensional changes caused by temperature differentials, degradation of mechanical properties of materials with rise of temperature, and degradation of element by cracking, crushing and reinforcement yielding, are considered. Formulation of failure envelop proposed by Küpfer and Gerstle [1] is adopted as illustrated in Fig.2. Within this model the initiation of cracking or crushing at any location occurs when the concrete principal stresses reach one of the failure surfaces. After the initiation of cracking in a single direction, the concrete is treated as an orthotropic material with principal axes normal and parallel to the crack direction. Upon further loading of singly-cracked concrete, a second set of cracks can be formed in the direction normal to the first set of smeared cracks. After crushing the concrete is assumed to lose all strength and stiffness. The stress-strain relationship of concrete at elevated temperature specified by EC4 [2] is adopted in this study.

The reinforcement is modeled using equivalent steel layers with constitutive properties solely in the direction of reinforcement. Bonding between the steel layer and concrete layers is assumed to be perfect. The stress-strain curve of reinforcing steel at elevated temperatures by EC4 [2] is used.

NUMERICAL ANALYSIS OF SOLID REINFORCED CONCRETE SLAB IN FIRE

In order to demonstrate the reliability and accuracy of the formulation described above to model reinforced concrete slabs in fire, reference is made to a rectangular solid reinforced concrete slab tested in fire by Lim et al [3]. The details of the slab are shown in Fig. 3. A surface load of 5.4kPa was applied to all of the shell elements. This load was kept constant during heating.

The comparison of numerical predictions with test results are shown in Fig. 4. Clearly, finite element model can accurately predict the deflection of the tested slab, showing a high deflection rate during the first stage, followed by a gradual deflection due to commencement of tensile membrane action, finally increasing again after 140 min due to yielding of the steel re-bar.
CONCLUSIONS

In this research, a shell element based on corotational approach (CR) is developed for modelling reinforced concrete (RC) slabs under fire conditions. The geometric and material nonlinearities are considered. Complex structural slab behaviour in fire conditions, such as thermal expansion, cracking or crushing of concrete, and change of material properties with temperature are considered. CR-approach was used to describe geometric nonlinearity. The proposed shell element was used to model a solid slab tested in fire and good agreement with mid-span deflection was obtained.

REFERENCES


Study of Heated AFM Tip Surface Using Molecular Dynamics Simulation for Thermomechanical Data Storage

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INTRODUCTION

At present, the magnetic hard disk is the dominant method for storing data in the computer and IT industry. However, areal storage densities achievable by today magnetic recording technologies are limited to around 150-200 Gbits/in², owing to the well-known superparamagnetic effect (SPE).

In order to avoid the SPE barrier, the industry, in the past decade, has managed to push up the limit for more storage capacity and higher data transfer rates in economical, reliable, and efficient ways. A new technology, atomic force microscope (AFM) based thermomechanical data storage has been brought forward as one of the most attractive alternative solutions [1], [2].

Through combining MEMS technologies with AFM technique which has a resolution on the scale of single atoms, AFM-based data storage is expected to show its potential when the atomic-level bit can be read and written. In the AFM thermomechanical data storage, a resistively heated AFM microcantilever writes/reads a data bit by scanning over a media substrate, as shown in Figure 1.

In this new emerging recording technology, the behavior of heat and mass transfer during thermomechanical data bit formation process is a critical factor affecting the areal storage density and data bit writing/reading speed. However, this behavior is still not well understood. In this article, using molecular dynamics simulation (MD), the surface construction of an AFM tip is analyzed to provide a helpful foundation for the future study of heat and mass transfer.
MD METHOD

On the atomic scale, a solid material can no longer be considered as a continuum. Thus, conventional continuum mechanics is difficult to apply. On the other hand, the motion of individual atoms becomes dominant. In this study, MD method is used to simulate the deformation of an atomic lattice.

The Tersoff-type three-body potential [3] is employed to describe the interactions between silicon atoms. The total Tersoff energy, \( V \), of the tip model is expressed in terms of the summation of atomic pair interactions, as follows.

\[
V = \frac{1}{2} \sum_{i<j} W_{ij}
\]

where \( w_{ij} \) is the bond energy about all the atomic bonds, \( i, j, k \) label the atoms of the system, \( r_{ij} \) is the length of the \( ij \) bond, \( b_{ij} \) is the bond order term, \( \varsigma_{ijk} \) is the bond angle between the bonds \( ij \) and \( ik \), \( f_R \) represents a repulsive pair potential, \( f_A \) represents an attractive pair potential, \( f_C \) represents a smooth cut-off function to limit the range of the potential, and \( \zeta \) counts the number of other bonds to atom \( i \) besides the \( ij \) bond.

RESULTS AND DISCUSSION

The choice of time step length is very important to achieve an efficient MD simulation. Too short a time step, and the phase space of the system will be sampled inefficiently. In this article, the Gear’s five-value predictor-corrector algorithm [4] is used for the numerical integration of the equations of the motion of individual atoms.

To select the suitable time step length, three different time step lengths, \( \Delta t = 0.1fs, 1fs \) and 10fs, are tested in simulating and relaxing the AFM tip to its minimum energy configuration at 300K. Figure 3 shows the temperature variations during 6000~9000 steps with these 3 different time step lengths.

According to Figure 3, it is apparent that the system reaches equilibrium state with temperature \( T = 300K \) most quickly.

MD MODELING

In this article, an AFM tip with four-sided pyramidal shape shown in Figure 2, which is convenient to be constructed using anisotropic etching techniques, is used. The sloping face of the AFM tip is determined by a (111) plane in the Si crystal, well defined and yielding very sharp edges. The four adjacent diamond (111) planes form a \( 60^\circ \) pyramidal crystal structure where the point of intersection of the planes constitutes the AFM tip. Therefore, the tip maneuvers along the \( \{100\} \) planes that are parallel to the surface of the polymer substrate.

The height of the AFM tip is \( 5a=2.715nm \), consisting of 5 Si (100) unit cell layers stacked upon each other, where \( a \) is the lattice constant (\( a=0.543nm \)). There are 3x3 atoms in the bottom layer of the tip, which corresponds to a contact area of approximate 0.59 nm\(^2\) (\( \sqrt{2a} \times \sqrt{2a} \)). There are 13x13 atoms in the top atom layer of the tip, which corresponds to a contact area of approximate 21.23 nm\(^2\) (\( 6 \times \sqrt{2a} \times 6 \times \sqrt{2a} \)). The total number of atoms is 1384.
with time step $\Delta t = 1$fs compared with $\Delta t = 0.1$fs and 10fs. If the time step is 10fs, the conserved quantities in the simulation of the system, e.g., temperature, start to fluctuate wildly, eventually leading to catastrophic instability (‘blowing up’), as shown in Figure 3(b). This is because the time step is too long such that the trajectories of the atoms are extrapolated into regions where the potential energy is very high, for example if the atoms overlap. Thus, time step $\Delta t = 1$fs is used for the simulation of the AFM tip.

Furthermore, the reconstruction of the tip surfaces should be also considered due to the following factors. First, the dimer reconstructed surfaces are more stable than the original truncated crystal. Second, the surface construction affects the heat diffusion, which is important for analyzing thermomechanical data bit formation. Finally, the adhesion will occur because of the bonding between the tip and substrate, if the reconstruction phenomenon is not considered.

Parts of the surface atoms in the first layer are calculated and monitored, as shown in Figure 4, in which Figure 4 (a) and (b) schematically illustrate the atom locations before and after reconstruction, respectively.

The surface atom displacements are displayed in Figure 4 (b) after reconstruction when the system reached equilibrium state. According to this figure, the simulation result does not display a regular (7x7) reconstruction surface as the bulk material surface. This phenomenon is consistent with two actual factors: the tiny size and pyramidal structure of the AFM tip. Due to the tiny size of the AFM tip, the atoms in different surfaces interact with each other, so as to affect their displacements. Furthermore, unlike general bulk silicon, the AFM tip is pyramidal structure. The atoms in different height levels have different potentials, so they are exerted by different forces and move with different displacements.

**CONCLUSIONS**

The MD simulation method for the AFM tip has been proposed, including the Tersoff potential and the Gear’s five-value predictor-corrector algorithm. According to simulation test, a suitable time step, 1fs, is determined.

With the MD simulation, it is found that the simulation result of the atom reconstruction on the AFM tip surface does not display a regular (7x7) reconstruction surface as the bulk material surface, which means that the parameters of the bulk material cannot be used to analyze the properties of the AFM tip directly. The MD simulation results provide a helpful foundation for the future study of heat and mass transfer in the AFM tip during thermo-mechanical data bit formation. After the reconstruction simulation, subsequent simulations can be initiated with this equilibrated structure to simulate the heat transfer in the AFM tip.

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INTRODUCTION

Ionic polymer-metal composite (IPMC) is a type of wet electro-active polymers (EAP) which consists of a thin polyelectrolyte membrane and a type of noble metal chemically plated on both sides of the membrane. IPMC can undergo a fast and large bending motion when a low electric potential is applied to its electrodes. Conversely, IPMC can generate a measurable electric potential when it is subject to a sudden bend. One of the important applications of IPMCs in the area of biomedical instruments which contact with human organs or tissues. In this article, an analytical model is developed to depict the dynamic response of a simply-supported IPMC beam resting on human tissues under an alternative electric field. A closed-form solution is obtained to describe the transverse vibration of the IPMC beam. Based on this solution, the beam deflection curve is obtained and the pressure generated on human tissues is calculated by numerical integration.

MODELING OF IPMC BEAM

As shown in Figure 1, a simply-supported Nafion based IPMC beam of length \( L \), thickness \( H \) and width \( b \) is studied. The IPMC beam is chemically plated with platinum electrode on both sides and \( x_1 \) and \( x_2 \) are the coordinates of the two ends of the electrode. The thickness of Nafion part of IPMC is denoted as \( h \). The effect of tissues is modeled as a Winkler foundation with stiffness \( k \). It is assumed that the alternative electric potentials at the top and the bottom surfaces of the IPMC beam are \( \phi_0 e^{j\omega t}/2 \) and \( \phi_0 e^{j\omega t}/2 \), respectively, where \( \phi_0 \) is the time-independent magnitude of electric potential, \( \Omega \) is the angular frequency and \( j \) is the imaginary unit. Due to the redistribution of cations and associated water under the applied electric field, the IPMC beam will vibrate at the same frequency as that of the applied electric potential. The motion equation of the beam can be obtained from Euler-Bernoulli’s beam theory as

\[
y_l \frac{d^4 u_l}{dx^4} + \rho A \frac{d^2 u_l}{dt^2} + c_l \frac{du_l}{dt} + k u_l = \frac{d^3 M^e(x,t)}{dx^3}
\]

where \( y \) and \( I \) are the Young’s modulus and moment of inertia of the beam, respectively; \( \rho \) is the material density of the IPMC beam; \( A \) is the cross-sectional area; \( c_l \) is the viscous damping coefficient; \( u_l(x,t) \) is the transverse displacement; and \( M^e(x,t) \) is the bending moment due to the electric field which can be expressed as

\[
M^e(x,t) = \frac{M^e_0}{\pi} \left[ H(x-x_1) - H(x-x_2) \right] e^{j(\omega t - \delta_0)}
\]

where \( H(t) \) is the Heaviside function; \( M^e_0 \) is the time-independent amplitude of bending moment and \( \delta_0 \) is the phase difference between bending moment and applied electric potential. Using the method of separation of variables, solution to Eq. (1) can be derived as

\[
u_l(x,t) = \sum_{n=1}^{\infty} \frac{2n\pi M^e_0}{\rho A c^2} \left[ \cos \left( \frac{n\pi x_1}{L} \right) - \cos \left( \frac{n\pi x_2}{L} \right) \right] \sin \left( \frac{n\pi x}{L} \right) \frac{e^{j(\Omega - \delta_0) t}}{L^{3/2}}
\]

where \( \omega_n = \sqrt{\frac{Y_l - (m^2 \pi^2)/L^2 + k/\rho A}} \) is the natural frequency of the beam for zero damping; and

\[\delta_n = \delta_0 + \tan^{-1} \frac{\omega_n}{\omega_n^2 - \Omega^2}\]

is the phase lag due to the viscous damping.

With the solution of Eq. (3), the total pressure generated by the IPMC beam can be readily obtained by

\[
p = k \cdot \int_0^L u_l H(u_l) dx
\]

The computation of total pressure can be implemented by numerical integration.

BENDING MOMENT DUE TO ELECTRIC FIELD

Considering that the volumetric strains of clusters and hence the axial strain of IPMC are affected by the water uptake [1], the moment rate is expressed as,

\[
M^e(t) = \int_{-\infty}^{\infty} Y_w \left[ \frac{1}{3} \frac{w(z,t)}{1 + w(z,t)} \right] dz
\]

where \( Y_w \) is the Young’s modulus of polyelectrolyte membrane; \( w(z,t) \) is the function accounting for the water uptake in clusters; and the dot above all symbols denotes the first derivative with respect to time.

Integrating Eq. (5) over time domain with the initial conditions of \( M^e(0) = 0 \) and \( w_i(z,0) = w_c(z,0) = w_o \), and considering...
the thicknesses of anode boundary layer (ABL) and cathode boundary layer (CBL) [1], the bending moment can be obtained as

\[ M^\prime = -\frac{YbhL}{6(1+w_0)}[(w_A(t) - \beta w_c(t)) - (1-\beta w_0)] \]  

(6)

where \( w_A \) and \( w_c \) are the water uptake at ABL and CBL, respectively; \( \beta = L_c / L_A \), \( L_A \) and \( L_c \) are the thickness of ABL and CBL, respectively, which can be determined by integration of the normalized charge density under an electric field. The water uptakes \( w_A \) and \( w_c \) are governed by the following two equations [1],

\[ \dot{w}_A(z,t) = D_A \dot{t}_A \]  

(7)

\[ \dot{w}_C(z,t) = D_C \dot{t}_C \]  

(8)

where \( D_A \) and \( D_C \) are the constant coefficients accounting for the diffusion in ABL and CBL, respectively; \( t_A \) and \( t_C \) are the pressure applied on water within the clusters at ABL and CBL, respectively. Detailed calculation of \( t_A \) and \( t_C \) can be found in [1, 2].

Considering small strains in clusters during beam vibration and applying Taylor’s series expansion at \( w_0 \) to Eqs. (7) and (8), the approximate solutions of \( w_A \) and \( w_c \) can be obtained as

\[ w_A(z,t) = \exp\left[\int F_A(\xi) d\xi\right] \cdot \left[ C_1 + \int \left[ \exp\left(-\int F_C(\xi) d\xi\right) \cdot \left( F_A - F_A w_0\right) d\xi \right] \right] \]  

(9)

\[ w_C(z,t) = \exp\left[\int F_C(\xi) d\xi\right] \cdot \left[ C_1 + \int \left[ \exp\left(-\int F_C(\xi) d\xi\right) \cdot \left( F_C - F_C w_0\right) d\xi \right] \right] \]  

(10)

where \( C_1 \) and \( C_2 \) are the integration constants;

\[ F_A(w_A) = D_A(1+w_0)\partial\partial_t(w_A) \]

\[ F_A(w_A) = \frac{\partial F_A(w_A)}{\partial w_A} w_A = w_0 \]

\[ F_c(w_c) = D_C(1+w_0)\partial\partial_t(w_c) \]

\[ F_c(w_c) = \frac{\partial F_c(w_c)}{\partial w_c} w_c = w_0 \]

Using Eqs. (9) and (10) and considering initial conditions, the bending moment generated by the IPMC beam can be obtained as

\[ M^\prime = -\frac{YbhL}{6(1+w_0)} \left( \frac{f_A}{f_1} \left[ \exp\left(\frac{f_{11}}{f_1}\right) - 1 \right] \right) \]

\[ -\frac{\beta f_c}{f_1} \left[ \exp\left(\frac{f_{11}}{f_1}\right) - 1 \right] e^{j\pi\sigma_{-2}} \]  

(11)

Detailed expressions of coefficients in Eqs. (11) and (12) can be found in [2].

Substituting Eq. (11) into Eq. (3), the transverse vibration of the IPMC beam can be calculated. For the static signal input, the bending moment should be calculated by Eq. (12).

**VERIFICATION AND ILLUSTRATIVE EXAMPLE**

First, the bending moment solution of Eq. (12) will be validated by comparing the tip displacement of a cantilever beam with the results obtained in [1]. Considering a Nafion based IPMC beam in Li\textsuperscript{+} form with length 18 mm, width 2 mm and thickness 224 µm, the plating metal is platinum and the thickness of electrode is 6 µm for both top and bottom surfaces of the IPMC beam. It is assumed that the initial water uptake of IPMC is \( w_0 = 0.533 \) and the electric potential is 1-volt DC signal. For simplicity, the viscous damping of the IPMC is set as zero. Figure 2 shows the comparison of normalized tip displacement from this model and the results in [1].

**Figure 2. Comparison between calculated displacement and data in [1]**

Subsequently, the DC potential is replaced by a 1-volt sinusoidal potential with frequency 0.25 Hz. The stiffness of tissues is set as 15 kPa. By using Eqs. (11) and (3), the beam deflection curve at any specified time can be obtained. Figure 3 illustrates the IPMC beam deflection and the pressure distribution on human tissues when the electric potential is at its maximum. It can be observed that the maximum deflection of IPMC beam is in the order of 0.01 mm and the maximum pressure is around 120 Pa. The total pressure can be evaluated.
Stress Wave Propagation in Discontinuous Geological Media

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INTRODUCTION

Rock mass is a discontinuous medium which consists of various discontinuities in different scales and forms, such as rock faults, joints and fractures. These discontinuities significantly affect the rock mass mechanical behaviour, especially in dynamic loading conditions. Over the past decades, a number of theoretical approaches have been developed to describe the rock mass properties, e.g., the equivalent medium theory and the displacement discontinuity theory. However, they are mainly derived with respect to quasi-static loading conditions. More extensive theoretical and experimental studies are necessary for stress wave propagation in discontinuous rock medium.

In the present study, wave propagation theory and displacement discontinuity theory are applied to fractured rock mass. Split Hopkinson Pressure Bar (SHPB) tests are carried out to capture the wave propagation and attenuation across rock joints which are artificially constructed by using a dry, partially or fully saturated sand layer. Effects of the aperture and the moisture contents of the sand layer on the wave propagation in discontinuous granite bars are investigated. By analyzing the experimental results, it is found that the increase of the aperture and the moisture content of the sand layer will both attenuate the stress wave propagation across the rock joints. Meanwhile, the stress-strain relations for the sand layer joints are also determined by the SHPB tests which can be used for derivation of dynamic rock mass properties.

THEORETICAL MODELS

The general equation for one-dimensional elastic wave propagation is:

\[ \text{by Eq. (4), which is } 0.2298 \text{ gram for this case. This pressure is equivalent to } 0.4693 \text{ mmHg on the human tissues beneath the beam.} \]

CONCLUSIONS

A dynamic IPMC beam model on human tissues was developed. Explicit bending moment expressions were derived for both dynamic and static electric potentials. Based on the bending moment expressions, the beam deflection curve and the pressure distribution generated on human tissues were obtained. The developed model is useful not only for IPMC related biomedical instruments interacting with human tissues but also for any other devices that utilize IPMC materials.

REFERENCES


**Figure 3.** (a) Deflection and (b) pressure distribution of IPMC beam under 1 volt sinusoidal potential \((k=15 \text{ kPa})\)
\[
\frac{\partial^2 u}{\partial x^2} = \frac{1}{c^2} \frac{\partial^2 u}{\partial t^2} 
\]

... (1)

where \( u \) is the displacement of an elastic material, and \( c \) is the longitudinal wave velocity of the medium under concern. The displacement variable can be expressed by

\[
u(x,t) = u_1(t - \frac{x}{c}) + u_2(t + \frac{x}{c})
\]

... (2)

In the displacement discontinuity theory, the stress field across a joint is assumed to be continuous and the displacement field is not, that is,

\[
\sigma'(x_1,t) = \sigma'(x_2,t)
\]

\[
\Delta u(x_1,t) = \int_0^l \nu'(x_1,t) dt - \int_0^l \nu'(x_2,t) dt = \frac{\sigma(x_1,t)}{k}
\]

where \( x_1 \) represents the location of fracture, and the subscript – and + means in the left and right sides of a joint, respectively. \( k \) is the specific stiffness of fractured rock mass.

**EXPERIMENTAL STUDIES AND RESULTS**

To simulate the wave propagation across discontinuous medium SHPB pressure bars made of good quality granite which is cored from an underground construction site are used. Single and multiple sand layers with different apertures and moistures are used to represent discontinuities in the granite medium. The granite bars and the sand layer in the experiments are shown in Figs. 1 and 2. A schematic diagram of the SHPB tests is depicted in Fig. 3.

It is seen from Fig. 3 that there are two channels specially assigned for the strain gauge at the front of the incident bar, which are used for the calibration of the incident wave. The pulse wave is generated by a hand hammer. Fig. 4 shows the typical incident and transmitted waves output from the gauges.

The ratio of the peak values of the transmitted wave to the incident wave with different moisture contents and apertures of the sand layer are shown in Fig. 5. As can be seen, the wave transmission ratio will decrease with the increasing of the moisture content and the aperture of the sand layer. It gives quantitatively the effect of the discontinuous sand layer and water content on the stress wave propagation. Figs. 6 and 7 show the stress-strain relations of the discontinuous layer. It is seen that the aperture affects significantly the stress-strain relation of the sand layer.
Fig. 5  Effect of moisture content on the ratio

Fig. 6  Stress-strain curve for dry sand

Fig. 7  Stress-strain curve

(for 10% moisture sand joint)  
(for 4mm thickness sand joint)
An Application of Supply Chain Optimization for Marine and Offshore Construction

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INTRODUCTION

An optimization to minimize subcontracting costs was performed for an internal supply chain of a major jack-up oil-rig producer. The underlying model for this application is based on the work in Teo (2006), which considers tactical planning for make-to-order (MTO) environment. The model assumes job shop flow within a MTO environment with fixed delivery lead time, which are characteristics typically found in the construction of marine vessels and offshore structures. This article describes an application of the model to the construction of sophisticated mobile offshore drilling rigs and illustrates that the model is able to capture the essentials of offshore rig construction planning.

The key tactical planning parameters in the MTO production environment are the planning windows and the planned lead times. The planned lead time (PLT) is the key input for Material Requirements Planning (MRP) processing and is used to project how long a job would progress through the production stages. The production planner would use this projection to set the job release times and to monitor the actual progress of individual jobs. The planning window is the time difference between the delivery lead time and the planned lead time of production; a longer planning window allows more smoothing of the Master Production Schedule (MPS). The model in Teo (2006) determines the first two moments of production requirements as well as the expected queue lengths, which are performance measures to assess the appropriate setting of these planning parameters; refer to Teo (2006) for details of the model.

The data presented in this article has been disguised to protect the company’s confidential data but the insights drawn are identical to the conclusions based on the actual data.

BACKGROUND OF SUPPLY CHAIN

A typical hull built by the oil-rig producer is made up of steel structures called blocks. Each steel block is in turn constructed by joining the smaller structures called panels. A block commonly consists of ten to sixty panels. The steel panel represents the most elementary structure in the hull construction. A panel is built by first welding the steel plates together, and then by fitting outfitting components to strengthen the structure. This case study focuses on the operations planning for the panel production.

In the context of the model, the parts are classified into product families, namely the Big Panels, Small Panels and Outfits. Big Panels are defined as panels built by joining two or more steel plates while Small Panels are built using a single steel plate. Outfits are the outfitting components welded onto the panels. Jobs for the product families take different process routes through the various stages of its internal supply chain. Figure 1 shows the simplified version of the process routes.

The panel production is physically housed in three separate shops, namely the Blasting Shop, NC Shop and Panel Shop, which are physically located next to each other. The Blasting Shop consists of the Blast station, the NC Shop comprises of the two NC cutting stations and the Panel Shop includes the rest of the processing stations (mainly the welding stations).

CHALLENGES IN PRODUCTION PLANNING

Production control in the panel production is based on the planned lead times (PLTs) at the shop level. The PLTs for the Blasting Shop, NC Shop and Panel Shop are 5, 8 and 13 days respectively, which gives a total delivery lead time (DLT) of 26 days. Therefore all jobs are released 26 days before their due-dates, regardless of the product family that the job is from; that is, Small Panels and Outfits are released 26 days before their due dates, even though they require fewer processing steps than the Big Panels. Hence there might be opportunities to smooth the MPS by creating planning windows for the two product families.

The main challenge in panel production planning is the large amount of variability that exists in the system. First, there is a lack of coordination between production schedules of panels for different oil-rig projects. As a result, the aggregate internal demand for the panels is highly fluctuating. Second, raw steel plates of required thickness and grades are frequently unavailable and this delays the release of some panels. Third, the raw steel plates are stacked to conserve space and therefore picking a required plate at the bottom of
a stack can take substantial time, thus adding considerable variability to the picking time of the raw plates.

As a result, the final MPS that is adjusted to these sources of variability is highly fluctuating which leads to a highly variable production requirement for the production capacity. Furthermore, there are diverse processing requirements at the production stages due to the different cut lengths of plates as well as the different number of plates and outfitting components required for each panel. In case of this varying workload, there is a frequent shortfall in capacity. Jobs are subcontracted to vendors, who are located close by, if they are anticipated to be late based on the PLTs. The management aims to better utilize the in-house capacity and reduce the subcontracting costs.

The objectives of this case study are:

- To separate the schedules of the three product families into three individual MPS. This would enable the smoothing of the MPS for the individual product family, especially for the Small Panels and Outfits.

- To determine the optimal planning windows and PLTs of the individual stages with the objective of minimizing the total subcontracting cost. The DL T is fixed for the analysis as the company does not wish to change the current DL T which would affect the production schedule for the downstream stages. Thus the objective was to find the optimal allocation of the DL T for the planning windows and the PLTs.

The analysis does not take into account the work-in-process inventories. This is because the raw plates intended for an entire oil-rig project are typically purchased and stored in the warehouse before production commences. Thus the material cost is regarded as a sunk cost and is thus inconsequential to how the plates are scheduled through the production system. Furthermore, the value added to the jobs through processing was found to be significantly lower than the subcontracting cost. Hence the holding cost for the work-in-process inventories are ignored in the analysis.

**OPTIMIZATION**

The model was built for a network of 3 dummy stages (one for each product family) and 10 production stages. The inclusion of the dummy stage in the production network is to model the smoothing of the MPS. To characterize the production random variables, the production requirements are assumed to be normally distributed, which is consistent with the actual data. The demand is defined in terms of units per day and capacity in hours per day.

The nonlinear optimization program was formulated with the objective to minimize the total expected subcontracting cost and the decision variables are the planning windows and PLTs:

\[
\text{Min } \sum_i c_i (P_i - M_i)^+ \\
\text{s.t. } \sum_i \omega_k n_i + W_k - L_k = 0, \quad \forall k \\
W_k \geq 1, \quad \forall k \\
n_i \geq 1, \quad \forall i
\]

The notation \( c_i \) is the subcontracting cost per hour for stage \( i \), \( M_i \) is the nominal capacity for stage \( i \) (hours per day), \( P_i \) is the production requirement at stage \( i \), \( \omega_k \) is the number of times a job from product family \( k \) visits stage \( i \), \( n_i \) is the PLT of stage \( i \), \( W_k \) is the planning window for product family \( k \) and \( L_k \) is the DL T for product family \( k \) (which equals 26 for all \( k \)).

In the objective function, \( x^+ \) implies \( \max(x,0) \) and therefore \( \mathbb{E}[P_i - M_i]^+ \) represents the expected amount of expediting work per day. The normality assumption allows us to solve the objective function by the normal linear loss integral. The mean and variance of \( P_i \) are required to compute the loss integral, which are obtained using the model in Teo (2006). The first set of constraints defines the relationship between the families’ planning windows, PLTs and the DL T. The second and third sets of constraints assures that the planning windows and PLTs are at least the one-day planning time bucket, which is also the minimum duration that the management perceives to be appropriate in order to keep up with the highly dynamic system.

The objective function is conjectured to be convex and a plausible argument is produced here for this conjecture. The expected amount of expediting work at stage \( i \), \( \mathbb{E}[P_i - M_i]^+ \) is smaller with smoother production requirements, which in turn depends on the planning windows as well as the PLTs of stage \( i \) and all its upstream stages. Longer planning windows and PLTs result in smoother production requirements. Furthermore, as the amount of smoothing increases, there are decreasing returns in terms of less variable production requirements. To establish the convexity of the objective function, the Hessian matrix \( U(n) \) associated with the expected expediting work was considered, which is given by

\[
U(n) = \frac{\partial^2 f(n)}{\partial n n^T}
\]

where \( f(n) = \sum_k \mathbb{E}[P_k - M_k]^+ \) and \( n \) is the vector for the PLTs (every element of \( n \) is positive). Equation (1) has not been explicitly analyzed for the model. But based upon the above argument for convexity in terms of the relationship between the PLTs and the expediting cost, the second partial derivatives of \( f(n) \) with respect to \( n \) are positive, i.e. each element of \( U(n) \) is greater than 0. This entails that \( n^T U(n) \) is greater than 0, which is the sufficient condition for \( U(n) \) to be positive definite and \( f(n) \) strictly convex. Furthermore, as the constraints are linear as well, the optimization program is convex which ensures a global optimum solution. The convex nonlinear program was solved using the optimization toolbox in MATLAB which employs the Sequential Quadratic Programming method.
SUMMARY OF RESULTS

The optimal solution suggests that there should be a substantial amount of smoothing for the release of Small Panels and Outfits, with planning windows equal to 10.8 and 9.8 days respectively. However the planning window for Big Panel equals to 1.0 which indicates that there should be no smoothing of its MPS, and more days should be allocated to the PLTs to smooth production at the production stages.

The optimal PLT of the Blast station is 1.0, i.e. binding to the second set of constraint, and is much shorter than the current PLT of 5 days. A large proportion of its present PLT acts as safety lead time to buffer against the uncertainties of unavailable steel plates and the long picking times. The solution suggests a reduction of the PLT of Blast to 1.0 day. For the Small Panels and Outfits, most of the excess days from this reduction are reallocated to the planning windows. At the Join station, the optimal solution for the PLT stands at 7.8 days, which is the longest among the workstations. This is because the utilization at the Join station is high at more than 90% and its variability of its effective processing time is also relatively higher with a coefficient of variation of 0.76. Thus the Join station requires a relatively longer PLT to smooth its production. Likewise for the same reasons, the optimal PLTs of the Outfitting (Big Panels) and Outfitting (Small Panels) are also relatively long with each equal to 6.6 days.

The model assumptions and its predictive capability were validated using actual production data which proved to be reasonable. Furthermore the company is satisfied with the potential cost savings based on the above recommendations.

CONCLUSIONS

This project has already brought several benefits to the company. It has led to a greater awareness of the importance of production smoothing within the organization. One outgrowth from this case study is a project that looks into the estimation of workload for every panel so as to better regulate the job release. The project has also emphasized to the management the importance of steel plate availability and coordination between the construction schedules for different oil-rigs. There are now projects carried out to induce these improvements.

At present, the recommendations are yet to be implemented as the facilities of the yard are currently undergoing major changes. For instance, the company has recently purchased an automated welding machine for the Join station which would significantly change its capacity level. Once the system becomes steady again, new data would be collected for the model inputs to re-generate a set of recommendations for implementation.

REFERENCE


A Web-based Decision Support System for Route Planning in Liner Shipping

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INTRODUCTION

Liner shipping provides regular service between ports according to timetables published in advance. The services are open to all shippers, just like a bus line. There are five key functions in long-term liner shipping operation, namely, customer relationship management, market monitoring, cost management, service route planning and ship scheduling. For liner shipping, route planning and scheduling are very important as once the service route is determined, it will be difficult to alter it in the short run. Therefore route planning along with the feasibility study of the route must be done carefully to ensure the feasibility of the route in the long-run.

The proposed decision support system (DSS) aims to help players in liner shipping industry shortening the time needed for the service route planning. The DSS consists of two main modules, namely the scheduling and the financial analysis modules. The scheduling module finds an optimal proforma schedule (PFS) whereas the financial analysis module examines the financial consequences of the respective PFS in the long run.

OVERALL STRUCTURE

The DSS is web-based, therefore it can be accessed by more
users and data collection can be carried out faster. Figure 1 shows the proposed overall structure. Being a web-based system, the DSS includes three tiers. The first tier is the interface that interacts with the user. The second layer is the mechanism which handles all the calculations and optimisations. The last tier is the database that stores all the necessary information for processing. VB.net is used to develop the front end of the system, whereas in the second layer, Visual Basic is employed to develop mechanisms to perform simple manipulations of the records and interact with database as well as the optimisation and financial analysis modules which are developed in AIMMS. The database, on the other hand is mainly handled by Microsoft software. We use Access and Excel to keep all the necessary information.

THE SCHEDULING MODULE

The main objective of the scheduling module is to find an optimal PFS, which is basically a list of the port calls (the service route) along with schedule for arrivals and departures at all ports. The scheduling module has two modes, namely manual and automatic modes. For many existing ship liners, especially the medium-sized companies, they usually develop the PFS based on experience. To ensure a smooth implementation of the system, the DSS accommodates this by providing the manual mode. In this mode, mechanisms to do manual manipulations on the PFS such as port additions, deletions, insertions, changing the ports sequence and editing of port information are provided. In the automatic mode, an optimisation model (developed based on the Travelling Salesman Problem) is employed to find the optimal PFS.

THE FINANCIAL ANALYSIS MODULE

The financial analysis module is developed to allow the ship liners to examine the financial consequences of a PFS. The financial analysis module consists of five main sub modules, namely, fixed cost estimation, profit and loss analysis, cash flow table generation, sensitivity analysis and an optimisation model.

The fixed cost estimation sub module estimates the total fixed costs for the whole voyage. The profit and loss analysis sub module on the other hand calculates the profit (or loss) of the PFS for the whole voyage and annually based on the estimated loading at all ports (received from agents). In the cash flow generation sub module, the weekly inflow and outflow of cash are calculated based on the data from the two previous sub modules. In the sensitivity analysis sub module, the effect of varying certain parameters on the annual profit (or loss) is shown. In the last sub module, an optimisation model is provided. The model determines which pairs of port of loading (POL) and port of discharge (POD) that will give the maximum total contribution (total revenue – total variable costs). The decision variables are estimated loading between pairs of POL and POD, and the constraints are total moves at each port and the accumulated load when leaving each port.

CONCLUSIONS

In liner shipping, service route must be planned carefully to ensure that it is feasible in the long-run. The proposed DSS aims to help the ship liners to shorten the time needed for service route planning. The system is web-based so that it can be accessed by more users and allows faster data collection. The system consists of the scheduling and the financial modules; the first module provides an optimal PFS whereas the latter examines its financial consequences in the long run.

ACKNOWLEDGEMENTS

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INTRODUCTION

Artificial neural network (ANN) is a mathematical or computational model for information processing based on biological neural networks [1]. An ensemble NN usually includes several component networks in its structure, and each component network commonly uses a single feed-forward network. The design of ensemble NNs architectures can be formulated as a challenging multi-criterion optimization problem, since the architecture of ensemble NNs has a significant impact on a network’s generalization ability. In this paper, an ensemble NN, which combines the component networks by using the Akaike information criterion (AIC), is proposed. The AIC based ensemble NN searches the best weight configuration of each component network at first, and then uses the AIC as an automating design tool to search the best combining weights of the ensemble NN. One analytical function – the peak function is used to assess the accuracy of the proposed ensemble approach. The computational experiments have verified that the proposed AIC based ensemble NN outperforms both the simple averaging ensemble NN and the single NN.

DESIGN ENSEMBLE NEURAL NETWORKS USING AIC

In an ensemble network, there are a finite number of NNs. Each NN in this ensemble NN is referred to a component network. Usually the component networks in an ensemble NN are trained for the same task independently and their solutions are combined. A typical architecture of the ensemble NN is shown in Figure 1.

In general, an ensemble NN is constructed in two parts: creating component networks and combining component networks in the ensemble NN. The major steps of the proposed AIC based ensemble NN are shown in Figure 2.

For creating component networks, good regression or classification component networks must be both accurate and diverse. In this research, back-propagation neural networks (BPNNs) are adopted. Even with the same hidden nodes and same training data, the initial weights used randomly in the program may result in different performance of the same NN. So, the proposed method is focused on choosing each component network with the best initial structure first, then balancing their contributions to the ensemble NN.

Considering the network’s accuracy and the model’s complexity, the AIC is used to combine these best component networks. The AIC is an information criterion for the identification of an optimal model from a class of competing models. The AIC belongs to the indirect approaches because it penalizes the model complexity [2]. Since the model with the smallest AIC value is the optimal model, the weights should be in inverse proportional to the AIC values. By assuming the sum of the weights equal to one, and each weight is not more than one, the AIC-based weights of the component networks can be found.

COMPUTATIONAL EXPERIMENTS

To verify the performance of the ensemble NN proposed in this paper, the theoretical function — the peak function was applied. The peak function, which is shown in Figure 3, is a typical complex two-dimensional function used as demonstration in MATLAB as following:

\[
Z = 3(1 - x)^2 e^{-x^2 - (y+1)^2} - 10(x/5 - x^3 - y^5) - x e^{-x^2 - y^2} - e^{-(x+1)^2 - y^2} / 3
\]
The peak function with normally distributed noise (mean 0, variance 0.05) is adopted to perform the AIC based ensemble NN.

First, 11x11 evenly distributed data along both the x-axis and the y-axis are selected from the domain [-3, 3] as the training data for the simulation. Another 10x10 evenly distributed points from the same domain are used as the testing data. The maximum training epoch of each component network is set to 30. For comparison purpose, a simple averaging ensemble NN that combine the component networks with the uniform weights and a single NN are also adopted to solve this problem. The optimal number of the hidden nodes is selected as 13 by the trail and error method. Therefore, the single NN uses 13 hidden nodes in its hidden layer. In the other two ensemble NNs, there are 3 component networks with 7, 10 and 13 hidden nodes respectively. Each component network was trained 5 times randomly to find the best weight configuration of this component network. Then, the AIC value of each best component network was calculated to determine the component weights in the ensemble NN.

The statistical performance of the MSE on the training data set and testing data set of 20 runs for these 3 different methods are shown in Table 1. From Table 1, it can be observed that the proposed AIC method can obtain better results than the results by the other two methods. Thus, the result of this example demonstrated that the AIC based weighted ensemble NN outperforms the single NN and simple averaging ensemble NN.

**CONCLUSIONS**

Determination of model complexity in an NN is crucial in NN design. This paper aims to use AIC to balance the model complexity with model accuracy. Using AIC to combine these best component networks enables us to balance the ensemble network’s accuracy against the model’s complexity, and helps to create a simple, accurate and stable ensemble NN. In this paper, it can be found that the proposed AIC based ensemble NN outperforms other methods. Although the proposed ensemble NN performs well, there is still some space to enhance the performance of the proposed method. AIC can identify an optimal model from a class of competing models. In future works, the authors will focus on adopting AIC to determine automatically the number of hidden nodes in each component network and the number of the component network in an ensemble network.

**REFERENCES**


INTRODUCTION

Geomaterials exhibit significant changes for their responses to dynamic versus static loadings. Such noted changes are mainly involved in macroscopic properties. The behaviors of geomaterials at low strain rates are well understood, but their behaviors at high strain rates are still poorly understood [1]. To study the behaviors of geomaterials at high strain rates, various experimental tests have been proposed, such as the Charpy impact test and the well-known drop weight impact test. More recently, the Split Hopkinson Pressure Bar (SHPB) was proposed. The SHPB could obtain precise measurements of forces and displacements. The improved Hopkinson bars can be used for dynamical tests of specimens subjected to uniaxial tension, torsion, and simultaneous torsion and compression.

Discontinuous Deformation Analysis (DDA) is a new discrete element method presented by Shi Genhua in 1988 [2]. All the external forces acting on each block satisfy the equilibrium equation; external forces and block stresses of each block achieve equilibrium; and the deformation behavior of block is described by the constitution equation. Furthermore, the penetration of any one block into the others is not allowed.

METHODOLOGY

The key and basic idea of DDA is that the displacements of any point within one block are determined by a complete one order approximation function [2]:

\[
\begin{bmatrix}
\Delta u \\
\Delta v \\
\end{bmatrix} =
\begin{bmatrix}
1 & 0 & -(y-y_0) & (x-x_0) & 0 & (y-y_0)/2 \\
0 & 1 & (x-x_0) & 0 & (y-y_0) & (x-x_0)/2 \\
\end{bmatrix}
\begin{bmatrix}
\Delta u \\
\Delta v \\
\end{bmatrix}
\]

All the potential energy of elastic strains, initial stresses, loadings, viscosity and inertia forces are computed. Minimizing the potential energy forms the global system equation:

\[
\begin{bmatrix}
K_{11} & K_{12} & \cdots & K_{1n} & D_1 & F_1 \\
K_{21} & K_{22} & \cdots & K_{2n} & D_2 & F_2 \\
\vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\
K_{n1} & K_{n2} & \cdots & K_{nn} & D_n & F_n \\
\end{bmatrix}
= \begin{bmatrix}
F_1 \\
F_2 \\
\vdots \\
F_n \\
\end{bmatrix}
\]

Solving the global system equation obtains the rigid movements and elastic deformations of blocks, thus, yields the strains, stresses of blocks.

For the SHPB, a compressive longitudinal incident wave \( \varepsilon_i \) is developed and travels along the input bar. Once the incident wave arrives at the bar-specimen interface, it splits into two waves: reflected wave \( \varepsilon_r \) and transmitted wave \( \varepsilon_t \). These strains have a relationship of \( \varepsilon_i + \varepsilon_r = \varepsilon_t \).

NUMERICAL RESULTS AND DISCUSSION

The material properties of steel input bar and output bar, and rock specimen are listed in Table 1. The velocity of the projectile is used as the impact force. The measure point A is on the input bar, the measure point B is on the output bar, and the measure point C is on the rock specimen. The stresses of input bar, rock specimen and output bar at given measure points are computed numerically and drawn in Figs. 1 and 2 for two rock specimens with elastic modulus 40Gpa and 20Gpa. The upper limit of time interval is chosen as 2.0e-6. Time step is 5100.

It can be observed that the incident wave, the reflected wave, and the transmitted wave are seen very clearly, and the relationship: \( \varepsilon_i + \varepsilon_r = \varepsilon_t \) can be satisfied for two rock specimens.

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Steel input bar and output bar</th>
<th>Rock specimens</th>
<th>Joint materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>density ( \rho ) ( \text{kg/m}^3 )</td>
<td>( 8000 )</td>
<td>( 2650 )</td>
<td>friction angle ( \alpha ) ( ^\circ )</td>
</tr>
<tr>
<td>elastic modulus ( E ) ( \text{Gpa} )</td>
<td>( 210 )</td>
<td>( 40 ) and ( 20 )</td>
<td>cohesion ( c_0 ) ( \text{Mpa} )</td>
</tr>
<tr>
<td>Poisson’s rate ( \nu )</td>
<td>( 0.3 )</td>
<td></td>
<td>tensile strength ( t_s ) ( \text{Mpa} )</td>
</tr>
</tbody>
</table>

CONCLUSIONS

The DDA is employed to simulate wave propagation in the SHPB. The good simulated incident wave, reflected wave, transmitted wave and their relationship demonstrate that the DDA is efficient for simulating the SHPB test although some techniques on the DDA should be done to improve the accuracy of stress waves.
INTRODUCTION

The rapid deterioration of aging steel infrastructures will require significant attention in developing new techniques for effective and economical revival of these structures. With the advanced mechanical properties, fiber reinforced polymer (FRP) materials have been successfully employed to upgrade and repair concrete structures in civil engineering [1]. Compared to the widespread use of FRP materials in rehabilitation of concrete structures, the application of FRP in strengthening steel members is not so well developed yet. Only in recent years are researchers beginning to investigate the feasibility of using FRP to strengthen and retrofit steel members [2-5]. In this connection, this paper investigates the effectiveness of FRP laminates in enhancing the flexural capacity of steel girders. Both static and cyclic experiments on FRP bonded steel girders under four-point bending are carried out. Different laminate types, bond lengths and strengthening configurations are employed in the experiment to investigate the influence of these parameters on the strengthening effect. The results are very promising as the FRP laminates are able to increase ultimate load capacity as well as stiffness of the steel girders.

EXPERIMENTAL PROGRAMME

Full-scale experiments on FRP strengthened steel girders are carried out at Heavy Structure Laboratory of Nanyang Technological University. The schematic of typical specimen configuration is shown in Fig. 1. All specimens are prepared and tested under four-point bending. Both damaged and intact steel girders are employed in the experimental programme. The experimental setup is shown in Fig. 2.

STRENGTH AND STIFFNESS

The relationships between load and deflection of the girders bonded with FRP laminates are illustrated in Fig. 3 and Fig. 4.
It can be seen that with different types of FRP laminates externally bonded, the steel beams get an increase of ultimate load from eight to nineteen percent. For the stiffness, the increase is from three to eighteen percent. The beam strengthened with CFRP laminate has the maximum ultimate load increase. Full-length bonding of CFRP laminate gives the maximum stiffness increase of eighteen percent to the steel girder.

**FAILURE MODES**

The steel beams strengthened with FRP laminates fail in a gradual process. For most specimens, top and bottom of the beam at midspan yield first when the load increases. After that, the load increases very slowly with the deflection going up continuously. Then the top flange begins to yield and deform where the loading point is. Finally, the bond failure happens when the plastic deformation of the beam develops to a very large extent. The photos of the typical failure mode of the strengthened steel beam are shown in Fig. 5.

**FORCE TRANSFER**

The variation of neutral axis and strain distributions in the laminate of the typical specimen are shown in Fig. 6 and Fig. 7, respectively. It is found that the deformation of the steel girder have significant effect on the strain development in the FRP laminate.
CONCLUSIONS

Adhesive bonding of FRP laminates is found to be a promising method to strengthen steel girders. In this study, the strengthening technique can produce a remarkable improvement in the ultimate load-carrying capacities of the strengthened beam girders as well as their bending stiffnesses.

![Fig. 7. Typical strain distributions in the FRP laminate of strengthened girder](image)

REFERENCES


Experimental Investigation of Free Spanning Submarine Pipeline Under Vortex Induced Vibration

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INTRODUCTION

A rigid riser mechanically clamped to an offshore wellhead platform experiences vortex induced vibrations due to the seawater wave and current actions. These vortex induced vibrations, if occurring at the natural frequency of the riser span, will induce large amplitude vibrations which may lead to fatigue failure at the girth welds of the riser pipe joints. To determine if the maximum allowable riser span length between each riser clamp is acceptable, the riser spans are usually analyzed using the same methodologies utilized for analyzing pipeline spans on the seabed due to undulating topography. The importance of damping effect on the boundary condition for spans has always been understated (DNV 1981). Gho et al (2005) performed simple calculations to emphasize on the effect of damping on the span length and showed that riser spans are sometimes limited by their clamping location. As a result, adjustable clamps have to be utilized, resulting in additional cost and installation time.

The usual design methodology for a seabed span design involves designing the pipeline span for fatigue analysis based on a set of S-N curves predetermined from the graph of amplitude normalized by pipe diameter against the reduced velocity experienced in the region. Based on the water current distribution in the region, the fatigue damage to the pipeline is accumulated through the Palmer-Milgren Method. Hence, the main requirement for the pipeline allowable span design is the data obtained from the vibration amplitude versus the reduced velocity graph. For the design of the riser span, the design methodology is based mainly on a “no motion” criteria. Hence, a more suitable basis for design will be experimental data based on the reduced velocity versus pipe stability parameter graph. This graphical representation is not new and has been utilized extensively in the design of heat exchanger generators.
EXPERIMENTAL SET-UP

The experimental program was carried out in the Hydraulic Modeling Laboratory of the School of Civil and Environmental Engineering, Nanyang Technological University. The experiments were performed in a 1.6m wide, 0.6m high and 21.0m long 2-step current flume. A section of the flume was modified so that the flume bottom was at constant elevation across the pipe length.

An acrylic pipe was suspended in the current flume with two vertical arms. The ends of the vertical arms were fabricated such that different boundary conditions could be represented in the experimental works. The plan and front views of the experimental set-up are shown in Figures 1 and 2 respectively. The final experimental set-up is shown in Figure 3.

Strain gauges, capable of measuring strains on acrylic material, have been used to measure the strain experienced in the acrylic pipe under vortex-induced vibrations. As the vibrations are dynamic in nature, the strain measurements were performed using a dynamic strain-meter, instead of the usual static strain-meter. Any 3-dimensional flow around the pipe boundary ends would affect the experimental results. An acrylic sheet at each end of the pipe boundary was installed to eliminate any possible flow disturbances.

The instrumentation performed for the experimental work included the following apparatus:

- One electromagnetic current meter to measure the mean current velocities in the horizontal direction.
- One dynamic strain-meter with analog data and recorder to measure the dynamic strain, which varies with time.

For the calibration of the current flume, the current velocity was measured across the cross section at different depths prior to the various sets of test so that information on flow uniformity could be obtained. During testing, the electromagnetic current meter was installed in an upstream location with respect to the acrylic pipe for the measurement of referenced flow velocities.

PRELIMINARY RESULTS

The dynamic strain experienced by the pipe in vortex shedding conditions against the measured period was obtained from the output of the dynamic strain-meter. The output for a 40mm diameter acrylic pipe with a span of 850 mm under fixed boundary conditions is plotted in the time domain graph and presented as Graph 1. To determine the strain within each given frequency band over a range of frequencies, the time domain graph is decomposed into the frequency domain using a Fast Fourier Transformation process. The frequency domain showed the spike in occurrences at the natural frequency of the acrylic pipe span at 5.18 Hz. Earlier calculations performed to determined the natural frequency of the acrylic pipe showed that the natural frequency of the pipe is within the range of

![Figure 1: Plan View of Experimental Setup](image1)

![Figure 2: Front View of Experimental Setup](image2)

![Figure 3: Actual Experimental Setup](image3)

![Graph 1 – Dynamic Strains in time domain](image4)
CONCLUSIONS

The current experiment is successful in measuring the dynamic strains of acrylic pipe due to vortex-induced vibrations. The calculated natural frequency of the acrylic pipe is found within the range of the test data.

REFERENCES


INTRODUCTION

In recent years, due to the rapid development of the meshless methods such as the reproducing kernel particle method (RKPM), much works have been done in the coupling of the boundary element method (BEM) with the meshless methods [1]. One advantage of the coupling procedure is that the expensive meshless methods can be applied to only a small but critical sub-domain, while the BEM is used in the remaining part of the infinite or semi infinite domain. Since coupled methods are often employed for solving of problems with high accuracy requirement, adaptive refinement is an important topic within the research of coupled methods. The main objective of this article is to describe an adaptive refinement procedure for the coupled BE-RKPM.

THE COUPLE BE-RKPM

In this study, the directly coupling procedure developed in reference [1] as employed and the problem domain \( \Omega \) is partitioned into two disjoint subdomains \( \Omega^B \) and \( \Omega^R \) (Fig. 1) where the RKPM and the BEM are applied respectively. The boundaries of \( \Omega^B \) and \( \Omega^R \) are denoted as \( \Gamma^B \) and \( \Gamma^R \) respectively such that for \( \Omega^B, \Gamma^B = \Gamma^B_u \cup \Gamma^B_t \cup \Gamma^{BR} \) and \( \mathbf{u}_B = \mathbf{u}_B^R \) on \( \Gamma^R \) and \( \mathbf{t} = \mathbf{t}^B \) on \( \Gamma^B_t \). The displacements and tractions compatibility conditions along the interface boundary \( \Gamma^{BR} \) can be expressed as

\[
\Gamma^{BR} = \Gamma^{BR}_u \text{ and } \mathbf{t}^{BR} = \mathbf{t}^{BR}_u \text{ and } \mathbf{u}^{BR} = \mathbf{u}^{BR}_u
\]

where \( \mathbf{t}^{BR}_u \) and \( \mathbf{u}^{BR}_u \) are the tractions (displacements) along \( \Gamma^{BR} \) with respect to \( \Omega^B \) and \( \Omega^R \) respectively. For \( \Omega^R \), the displacements along \( \Gamma^R, \mathbf{u}^R \) can be expressed as

\[
\mathbf{u}^R = \left[ N^R \right]^T \mathbf{u}^R + \left[ N^C \right]^T \mathbf{u}^C
\]

where \( \mathbf{u}^R \) are the RKPM shape functions and the nodal parameters along \( \Gamma^{BR} \) respectively. The final equations system for \( \Omega^B \) can be expressed as [1]

\[
\mathbf{K}^R \mathbf{u}^R = \mathbf{f}^R + \mathbf{f}^{RB}
\]

For \( \Omega^B \), the known terms \( \mathbf{u}^B \) and \( \mathbf{t}^B \) and the unknowns \( \mathbf{u}^B_\Gamma, \mathbf{t}^B_\Gamma, \mathbf{t}^{RB} \) and \( \mathbf{u}^{RB} \) are interpolated as

\[
\mathbf{u}^B = \left[ N^B \right]^T \mathbf{u}_\Gamma^B, \quad \mathbf{t}^B = \left[ N^B \right]^T \mathbf{t}_\Gamma^B\]  (4a)

\[
\mathbf{u}^{RB} = \left[ N^R \right]^T \mathbf{u}_\Gamma^{RB}, \quad \mathbf{t}^{RB} = \left[ N^R \right]^T \mathbf{t}_\Gamma^{RB}\]  (4b)

\[
\mathbf{u}^{RB} = \mathbf{u}_\Gamma^{RB}\]  (4c)

where \( \mathbf{u}_\Gamma^B, \mathbf{t}_\Gamma^B, \mathbf{u}_\Gamma^{RB}, \mathbf{t}_\Gamma^{RB} \) are the nodal values corresponding to the terms \( \mathbf{u}^B, \mathbf{t}^B, \mathbf{u}^B_\Gamma, \mathbf{t}^B_\Gamma, \mathbf{u}^{RB} \) and \( \mathbf{u}^{RB} \) respectively. The final equations system is given by [1]...
By using the unit displacement method, the relationship between \( \mathbf{t}^n \) and \( \mathbf{a}^n \) can be written as

\[
\mathbf{t}^n = \mathbf{W}^n \mathbf{a}^n + \mathbf{i}^n
\]

where the matrix \( \mathbf{W}^n \) and the vector \( \mathbf{i}^n \) can be computed from the known terms \( \mathbf{N}^n \), \( \mathbf{K}^n \), \( \mathbf{f}^n \) and \( \mathbf{N}^n \), \( \mathbf{t}^n \) can be eliminated from Eqn. 6 for solving \( \mathbf{u}^n \), \( \mathbf{t}^n \), \( \mathbf{a}^n \) and then \( \mathbf{u}^n \).

**A posteriori error estimations**

The Z-Z approach [2] adopted for the *a posteriori* estimation for the RKPM solution. The estimated error norm \( [\mathbf{e}^n]_i \) is given by

\[
[\mathbf{e}^n]_i = \int_{\Omega} \left( [\mathbf{e}^n]_i \mathbf{D} - [\mathbf{e}^n]_i \mathbf{D} \right) d\Omega
\]

where \( \mathbf{e}^n \) and \( \mathbf{a}^n \) are the RKPM stress field and a recovered stress field respectively. \( \mathbf{a}^n \) obtained using an extraction procedure so that the recovered stress at node \( i \), \( \mathbf{a}^n \) is given by

\[
\mathbf{a}^n = \mathbf{\bar{a}}^n - \mathbf{K}^n \mathbf{u}^n
\]

The residual approach suggested by Chen et al. [3] is adopted for the RKPM solution. The estimated relative error norm contribution associates with the \( i \)-th node

\[
[\mathbf{e}^n]_i = \int_{\Omega} \left( [\mathbf{e}^n]_i \mathbf{D} - [\mathbf{e}^n]_i \mathbf{D} \right) d\Omega
\]

The residual approach suggested by Chen et al. [3] is adopted for the BEM solution. The residual norm \( [\mathbf{e}^n]_i \) is defined as

\[
[\mathbf{e}^n]_i = \int_{\Omega} \left( [\mathbf{e}^n]_i \mathbf{D} - [\mathbf{e}^n]_i \mathbf{D} \right) d\Omega
\]

At point \( p \), a more accurate recovery stress \( \mathbf{a}^n (p) \) can again be obtained such that [2]

\[
\mathbf{a}^n (p) = \mathbf{\bar{a}}^n (p) + \mathbf{\Phi}^n
\]

Note that the integral operators \( \mathbf{\Psi}^n \) and \( \mathbf{\Phi}^n \) involve boundary integration over the whole domain boundary. The estimated residual at point \( p \), \( [\mathbf{e}^n]_i (p) \) and the estimated residual norm for the \( i \)-th element, \( [\mathbf{e}^n]_i \) are given by

\[
[\mathbf{e}^n]_i (p) = [\mathbf{e}^n]_i (p) - [\mathbf{e}^n]_i (p)
\]

\[
[\mathbf{e}^n]_i = \int_{\Omega} \left( [\mathbf{e}^n]_i \mathbf{D} - [\mathbf{e}^n]_i \mathbf{D} \right) d\Omega
\]

**ADAPTIVE REFINEMENT PROCEDURES**

During the adaptive refinement analysis, the estimated relative error \( \eta_{\text{RKPM}} \) of the RKPM solution will be computed as

\[
\eta_{\text{RKPM}} = \frac{[\mathbf{e}^n]_i}{\mathbf{h}_{\text{RKPM}}}
\]

In Eqn. 15, \( [\mathbf{e}^n]_i = \left( [\mathbf{e}^n]_i \mathbf{D} - [\mathbf{e}^n]_i \mathbf{D} \right) d\Omega \) is the energy norm of the RKPM solution. \( \mathbf{h}_{\text{RKPM}} \) is a compensational factor for improvement of the efficiency of the adaptive refinement. The objective of an adaptive refinement is to achieve a solution with \( \eta_{\text{RKPM}} \) less than a user prescribed target \( \eta_{\text{target}} \) such that

\[
\eta_{\text{RKPM}} \leq \eta_{\text{target}} \Rightarrow [\mathbf{e}^n]_i \leq \eta_{\text{target}} [\mathbf{e}^n]_i
\]

The *global refinement indicator* \( \xi_g \) is defined as

\[
\xi_g = \frac{\mathbf{R}^n}{\mathbf{G}^n}
\]

where \( \mathbf{R}^n \) and \( \mathbf{G}^n \) are the energy norm of the RKPM solution and a recovered. \( \mathbf{R}^n \) is  the energy norm of the BEM solution. The estimated relative residual error of the BEM solution \( \eta_{\text{BEM}} \) is defined as

\[
\eta_{\text{BEM}} = \frac{[\mathbf{e}^n]_i}{\mathbf{h}_{\text{BEM}}}
\]

The global boundary refinement indicator

\[
\xi_g = \frac{\mathbf{R}^n}{\mathbf{G}^n}
\]

By imposing the condition of equilibration of \( \mathbf{e}^n \) among all the unknown terms, the local boundary refinement indicator

\[
\eta_{\text{BEM}} \leq \eta_{\text{target}} \Rightarrow [\mathbf{e}^n]_i \leq \eta_{\text{target}} [\mathbf{e}^n]_i
\]

The *local boundary refinement indicator* \( \xi_l \) is defined as

\[
\xi_l = \frac{\mathbf{R}^n}{\mathbf{G}^n}
\]

where \( \mathbf{R}^n \) is the energy norm of the BEM solution and \( \mathbf{G}^n \) is the estimated residual norm.

The objective of the adaptive refinement is to achieve a residual error less than a user prescribed target \( \eta_{\text{target}} \) such that

\[
\eta_{\text{BEM}} \leq \eta_{\text{target}} \Rightarrow [\mathbf{e}^n]_i \leq \eta_{\text{target}} [\mathbf{e}^n]_i
\]

Eqs. 22 leads to the global boundary refinement indicator

\[
\xi_g = \frac{\mathbf{R}^n}{\mathbf{G}^n}
\]
A value of $\xi_i^B \leq 1$ implies that the target accuracy is satisfied, otherwise, further refinement is needed. In case that $\xi_i^B > 1$, a local boundary refinement indicator is computed by first defining ith element energy norm $\|\mathbf{u}\|_i^2 = \int_{\Gamma_i} (\mathbf{u}^T : \mathbf{G} : \mathbf{u}) \, d\Gamma$ and then the estimated relative residual error for the ith boundary element $\eta_{BEM,i}$:

$$\eta_{BEM,i} = \frac{\|\mathbf{u}\|_i^2}{\eta_{target}^B}$$

From $\eta_{BEM,i}$, the local boundary refinement indicator for the ith element $\xi_i^B$ is given by

$$\xi_i^B = \eta_{BEM,i}/\eta_{target}^B = \frac{\|\mathbf{u}\|_i^2}{\eta_{target}^B}$$

A value of $\xi_i^B > 1$ implies that refinement for the ith boundary element is needed. The adaptive refinement procedure for the BEM can be summarized as following [3]:

(i) Retrieve $l_i$, the boundary element size from the existing mesh.
(ii) Compute $\xi_i^B$ and $\xi_j^B$ for $i=1,...,NBE$. NBE is the number of BE.
(iii) The adaptive refinement will be stopped if $\xi_i^B \leq 1$. Otherwise, the BE mesh will be refined according to steps (iv) and (v).
(iv) Identify all elements with $\xi_i^B > 1$. Subdivide them into 2 equal size elements.
(v) Check all the boundary elements, subdivide the longer element of the 2 neighbours elements if their length ratio is outside the range [0.25, 4].

**NUMERICAL VALIDATION**

In this section, numerical study on a classic benchmark problem is given (Fig. 2). Linear RKPM and linear BEM are used with uniform and adaptive refinements applied. The initial meshes used and the partitions of the problem domain are shown in Fig. 2. Uniform meshes are obtained by reducing the elements size and node spacing uniformly. For adaptive refinement, adaptive meshes are generated by applying the procedure described above and values for $\eta_{target}^R$ and $\eta_{target}^B$ are both set equal to 5% and 3 refinements were carried out. The adaptive meshes generated and the convergence history is shown in Fig. 3. It can be seen that under uniform refinement, the convergence rate of the solution is reduced significantly. However, when adaptive refinement is carried out, the convergence rate improves. As a result, it can be concluded that the adaptive refinement procedure can significantly reduce the effect of singularity.
Residual Strength Assessment of a Cracked Tubular K-Joint

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INTRODUCTION

The prediction of the ultimate strength of tubular joints is based on the design equations of uncracked joints. However, many offshore structures contain cracks caused by severe cyclic loading conditions in the open sea. These fatigue cracks propagate and increase in size resulting in the reduction of the joint capacity. Therefore, there is a need to develop a systematic procedure for estimating the static residual strength of cracked tubular joints in practice. The most widespread and successful assessment approach is the failure assessment diagram (FAD) approach. Based on this approach, BS7910:2005[1] gives guidance for assessing the acceptability of defects in welded structures. In the BS7910:2005[1] Annex B, a specific assessment procedure for tubular joints in offshore structures has been proposed recently. In accordance with this procedure, a fatigue cracked tubular K-joint under combined loads has been assessed illustrating the usage of this approach[2].

FAILURE ASSESSMENT OF CRACKED TUBULAR JOINTS

Tubular joints can be assessed using BS7910:2005[1] Level 2 FADs which is the normal assessment route for general application. It has two methods, Level 2A and 2B as shown in Figure 1. The equations describing the assessment line are given as follows:

Level 2A: \[ K_x = (1 - 0.14L_\sigma)\left[0.3 + 0.7\exp(-0.65L_\sigma)\right] \]  
Level 2B: \[ K_x = \frac{E_{\text{inel}}}{L_\sigma} + \frac{L_\sigma}{2E_{\text{inel}}} \] 

The application of Level 2B requires the knowledge of complete stress-strain curve; in particular the region around the yield point has to be available in a detailed manner. However, there are many cases where this information is not available to the users. Therefore, Equation (2) is applied to a number of materials to generate a material independent Level 2A lower bound curve shown in Figure 1, which is the more conservative curve.

REFERENCES

This method adopts the assessment curve which uses the ratio of the stress intensity factor $K_I$ to the fracture toughness $K_{IC}$ as the vertical axis, and the ratio of the applied load $P$ to the plastic collapse load $P_u$ as the horizontal axis. If the service point falls inside the assessment curve, the structure is considered safe, otherwise, the structure is deemed unsafe.

**A CASE STUDY OF CRACKED TUBULAR K-JOINT**

A tubular K-joint containing a surface crack at the crown is assessed in this paper. The geometrical parameters of this joint is shown in Figure 2, and the detail dimensions are tabulated in Table 1, where, $\beta = \frac{d}{D}$, $\gamma = \frac{t}{2T}$, $\zeta = \frac{E}{P}$.

The dimensions of the surface crack measured from experimental tests are shown in Figure 3, where $2c = 100$ mm and $a = 12.75$ mm.

According to the geometries of the joint and detail dimensions of fatigue crack, the finite element model is generated to calculate the Mode-I stress intensity factors [3] for this assessment and the load conditions are shown in Figure 4. From the finite element analysis, the highest stress intensity factor is found to be equal to 42.50 MPam$^{1/2}$.

**FAILURE ASSESSMENT OF CRACKED TUBULAR K-JOINT**

The safety of this tubular K-joint under combined loads is assessed according to BS7910:2005 [1] Level 2A. For offshore tubular joints, the plastic collapse loads for the cracked geometry are determined by reducing the plastic collapse loads for the corresponding uncracked geometry. The plastic collapse loads of uncracked K-joint can be calculated using the standard equations given in many existing codes of practice. The applied axial load is $P_a = 150$ kN and the in-plane bending moment is $M_{ai} = 13.5 \times 2.580 = 34.83$ kNm. The correction factor for axial loading is given by

$$F_{a} = \left[1 - \frac{\text{cracked area}}{\text{intersection length} \times t_0} \right] \frac{1}{Q_0} = 0.93$$

From BS7910:2005 [1], the corresponding correction factor for in-plane bending is given by

$$F_{bi} = \cos \left(\frac{\phi}{2}\right) \left[1 - \sin \left(\frac{\phi}{2}\right)\right] = 0.274$$

The fracture toughness $K_{IC}$ of the material is 171.39 MPam$^{1/2}$, the yield stress $\sigma_y = 352$ MPa, and the ultimate tension stress $\sigma_u = 493$ MPa. Then the two assessment parameters $K_r$ and $L_r$ can be calculated as

$$L_r = \frac{\sigma_y}{\sigma_u} \left[\left(\frac{P_a}{M_a} \frac{M_{ai}}{M_{ai}}\right)^2 + \left(\frac{M_{ai}}{M_{ai}}\right)\right] = 0.75$$

$$K_r = \frac{K_{r1}}{K_{r2}} = 0.25$$

Then, the assessment point of this cracked K-joint can be plotted as shown in Figure 5.

The service point is found to fall inside the BS7910:2005 [1] Level 2A curve, hence this joint is still safe. This conclusion agrees with the previous experimental results.
INTRODUCTION

The actual behaviour of non-seismically designed (gravity designed) structures when exposed to an appreciable level of earthquake ground shaking is a topic of increasing interest in low seismic regions, particularly concerning the building performance and risk management. Much of the existing experimental work has focused on representative structural components and connections under static cyclic loading, with emphasis on the behaviour under large load reversals. Only limited information is available on the behaviour of this category of structures when subjected to real earthquakes up to a moderate intensity. The present study aims to fill in this gap. Two RC frames, one representing a typical design without considering any seismic detailing requirements, another having the same main reinforcement but with modifications to the detailing in critical regions in conformance with general seismic requirements, were constructed and tested on the shake table facilities in the Protective Engineering Laboratory. The test programme and the main results are reported here.

TEST STRUCTURES AND TEST PROGRAMME

The test structures were two-storey moment resisting frames, constructed at 50% reduced scale. Fig. 1 shows the dimension and reinforcement details of the two test frames, namely frame FC of the original design (Fig. 1(a)), and frame FM of the modified design (Fig. 1(b)).
The concrete used in the construction of the test frames had a maximum aggregate size of 10mm. The average cylinder compressive strength was about 40 MPa. In order to satisfy the similitude requirements for the cross-sectional resistance, the same grade reinforcement as in the prototype design was used in the test structures with a 50% reduction in diameter. The longitudinal reinforcing bars in the test frames were T10 (10mm diameter) deformed bars and stirrup steel was R6 (6mm diameter) plain round bar.

The test set-up was arranged such that the two plane frames were tested side-by-side simultaneously. In-between the two frames a pair of couplers was employed that allowed free sliding between the two frames in the direction of the excitation, but supported each other against incidental out-of-plane movement. Displacement transducers and accelerometers were installed to measure the dynamic response of the test frames at each floor level. Besides, a number of steel strain gauges were installed on the reinforcing bars in potential critical regions at the construction stage and measurements were taken from these strain gauges during the tests. Several pairs of LVDTs were also installed at the top and bottom regions of the ground storey columns to measure the plastic hinge rotations. Fig. 2 gives an overview of the test set-up and the main instrumentation. It is also noted that additional masses were installed on the floor slabs in accordance with the similitude requirements.

The reference input ground motion was taken from the ground motion component N70W at Vina del Mar site during the 1985 Chilean earthquake. This ground motion record has the character of a long effective duration of about 40 seconds, and it also has a relatively flat plateau in the acceleration response spectrum in the period range of interest. In the actual tests, the time scale of the recorded acclerogram was compressed by a time scale factor of $\frac{1}{\sqrt{2}}$ to accord with the half scale test structures. Figure 3 depicts the input motion used in the tests.

The test programme was organized such that a series of tests with increasing intensity was performed step by step. At the initial stage and after each simulated earthquake test, a complementary random vibration test was performed to acquire vibration information for the analysis of the change of the dynamic properties of the structures during the course of the tests.

![Figure 3: Input ground motions](image)

SUMMARY OF MAIN TEST RESULTS

Figure 4 shows typical measured response time histories during a medium intensity test. It can be seen that the effective excitation duration was about the order of 40 seconds. The base-shear vs. roof displacement hysteretic loops from different tests are combined, as shown in Fig. 5, to illustrate the overall hysteretic behaviour.

The two test frames exhibited comparable performance during the entire course of tests up to a peak ground acceleration of 0.46 g. Both frames did not exhibit visible damage during the lower intensity test with a peak ground acceleration of 0.10g, and visible cracks started to appear during the 0.23g test. No significant strength degradation occurred during and after the 0.23g test, as can also be observed from the hysteretic loops in Figure 5. The undesired detailing in frame FC at and around the joints and the column bottom end regions did not cause apparent problems during moderate shaking. Marked strength and stiffness degradations only occurred during the final strong base shaking with a peak acceleration of 0.46g. It is worth noting that this magnitude of ground shaking is only possible in strong seismic regions.
TESTS ON FRP REPAIRED FRAMES

In order to evaluate the effectiveness of commonly used FRP repairing method in restoring the stiffness and strength of frame structures for seismic resistance, the damaged frames after the test of 0.46g were repaired by means of crack injection with epoxy and FRP wrapping in the concentrated damage regions.

The FRP repaired frames were then subjected to a similar series of simulated earthquake tests. Figure 6 shows the repaired frames on the shaker and a comparison between the natural frequencies, which serves as an indicator of the stiffness, for the repaired frames during the course of tests as compared to their counterpart before the repair.

Test results indicate that the repair was rather effective. As shown in Figure 6(b) the initial stiffness of the repaired frames generally resembled the initial stiffness of the original frames. The degradation of the repaired frames followed a similar pace as in the original frames with the increase of the base motion intensity.

Debonding of the FRP strips, especially in areas where only surface bonding was possible such as at the foot of the columns, was presumed to be a major threat to the effectiveness of the repair during inelastic response. This problem was indeed observed to surface during strong base shaking tests (0.46g and the final failure test). Nevertheless, the debonding did not appear to occur or cause any significant problem up to a moderate ground shaking with peak acceleration of 0.23g.

ACKNOWLEDGEMENTS

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INTRODUCTION

The deformation mode and the crushing stress of metallic cellular materials under dynamic compression are of great interest to researchers and engineers. Many impact tests have been conducted to investigate the dynamic behavior of cellular materials. However, different and conflicting conclusions about the loading rate or strain rate effect on the crushing stress have been drawn. To understand the mechanism of the dynamic crushing of cellular materials, numerical simulations by employing a mesoscale Voronoi tessellation model are carried out. The foam specimen used in the numerical simulation is sandwiched between a moving rigid wall and a stationary rigid wall. This approximates the situation in an impact test for cellular materials. The crushing stress is measured at the impact side as well as the distal side. Some important factors that may influence the nominal crushing stress are investigated. Loading rate effect on the stress-strain relation of the cellular material is clarified. Rate sensitivity of cellular materials is also discussed by comparing the results based on the mesoscale simulation with the results of a continuum model and a shock wave theory.

MESOSCOSCAL MODELING OF CELLULAR MATERIAL

Two-dimensional Voronoi diagrams are constructed to represent the cellular material. The Voronoi diagram is similar to the physical structure of foam materials. In many foaming processes, gas bubbles nucleate throughout the liquid metal and grow, initially as spheres but later as polyhedral cells when the bubbles start to interact. The deformation mode of the foam is primarily influenced by the impact velocity. In the current study, we catalogued three deformation modes, namely, random mode, transitional mode and progressive mode, corresponding to increasing impact velocities, as shown in Fig. 1 (a)–(c). Under low velocity impact, the deformation is firstly localized in one or two bands of collapsing cells, which are the weakest regions of the sample material. Then the crushing band begins to diffuse and then all the cells collapse until they are fully densified. Basically, the deformation is randomly distributed. Under moderate impact velocities, the inertia of the cells becomes important and the cells tend to collapse progressively starting from the impact side. Finally, when a foam material is impacted at a sufficiently high velocity, the deformation band is highly localized at the impact side (the deformation band appears within a cell). A crushing band initiated at the impact end and propagates along the impact direction. It is also called the shock wave phenomenon in the cellular material.

Fig. 2 gives the nominal stress at the impact end vs. nominal strain at different velocities. It is seen that initial peak stress and plateau stress (at the plastic region) increases with the increase of the impact velocity. It is also interesting that while the nominal stress has a strain-hardening trend in quasi-static condition, the nominal stress under high velocity impact appears to fluctuate over a constant value. The nominal stress at the stationary side under high velocity impact is so different from that at the impact side, as shown in Fig. 3. It is seen that the stress-strain curves at the stationary side are nearly similar no matter what the impact velocity is.
The loading rate effect on the nominal stress at the impact and stationary sides may be sourced by the micro-inertia of cell walls and strain rate sensitivity of the cellular material. It is difficult to differentiate the inertia effect and strain rate effect in a physical impact tests. The loading rate effect may thus be misunderstood as the strain rate sensitivity of the material.

The average value of the nominal stresses (over the strain range 0–0.8) at the impact end based on the mesoscale model and a continuum model are given in Fig. 4 and compared with the prediction by a shock wave theory. It is seen that the shock theory slightly overpredicts the stress at the impact end. The discrepancy may be result from the idealized RPPL model, which neglects the elastic deformation. The nominal stress at the stationary side can be derived as a constant based on the shock wave theory.

SUMMARY

The enhancement of the nominal stress at the impact side under dynamic conditions is primarily due to the micro-inertia of cell walls. It is important to exclude the inertia force from the measured nominal stress at the impact side. The nominal stress at the stationary side is found to be insensitive to loading rate unless the base material has strong rate-dependency. The appreciable rate-sensitivity of some cellular materials, e.g., ALPORAS, cannot be explained by the current study. One possible factor causing the rate sensitivity is that the entrapped air in the closed cells has an influence on the deformation of the cell wall. The entrapped air, however, is not modeled in the present study.

Strength of Slab-Column Connections under Gravity and Lateral Loadings

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INTRODUCTION

The shear strength of slabs \( v_c \) in the ACI 318-05 [1] is expressed by three equations, as shown in Equations (1) to (3), and it is determined from the lowest of the following expressions:

\[
\begin{align*}
\nu_c &= \left(1 + \frac{2}{\beta}\right) \sqrt{\frac{f_{\text{c}}'}{6}} \\
\nu_c &= \left(\frac{\alpha_d}{b_o} + 2\right) \sqrt{\frac{f_{\text{c}}'}{12}} \\
\nu_c &= \frac{1}{3} \sqrt{f_{\text{c}}'}
\end{align*}
\]

where: \( \beta \) is the ratio of the longer side to the shorter side of the concentrated load (or columns); \( \alpha \) is 40 for interior columns, 30 for edge columns, and 20 for corner columns; \( b_o \) is the length of critical shear perimeter taken at a distance of 0.5\( d \) away from the column face; \( d \) is the effective depth of slabs; and \( f_{\text{c}}' \) is concrete cylinder strength.

Although these equations can give reasonable prediction, it can be improved further. The use of three equations for \( \nu_c \) can be simplified by using only one equation. These equations also ignore the influence of slab flexural reinforcement, and hence, underestimate the shear strength when the slab flexural reinforcement ratio is high. In an attempt to improve the shear strength prediction for slabs, a simple equation was introduced in a paper on symmetrical punching by Teng et. al [2].
paper extends the applicability of the proposed simple equation to predict the shear strength of slabs with unbalanced moment transfer. A new equation to calculate the slab ultimate shear stress at the critical shear perimeter of slab-column connections under gravity load and unbalanced moment transfer is also derived and verified by experiments.

SLAB-COLUMN CONNECTIONS UNDER SYMMETRICAL PUNCHING

Proposed equation for punching shear strength

Teng et al. [2] proposed a simple equation for calculating the shear strength of slabs as a function of concrete compressive strength \( f'_c \), and reinforcement ratio \( \rho \). The proposed formula shown in Equation (4) had been verified against 238 slab data under symmetrical punching and was found to be more accurate than existing code formulas. Table 1 shows that the simple form of Equation (4) is more accurate than the three \( v_c \) equations given in Equations (1) to (3) by the ACI 318-05.

\[
v_c = 0.6\rho^{1.13}(f'_c)^{0.13} \quad \text{(MPa)} \tag{4}
\]

where: \( v_c \) = punching shear strength of slab-column connections; \( \rho \) = average flexural reinforcement ratio in percentage; \( f'_c \) = concrete compressive strength.

Shear stress calculation for rectangular columns

For rectangular columns, the ultimate shear strength calculated by assuming a uniform stress distribution along the critical shear perimeter, \( v_c = V/\beta d \), may result in a lower shear stress compared to the actual shear stress that develops along the shorter side of the rectangular column. Thus, a factor \( \beta_0 \) needs to be introduced into the shear stress calculation to consider column rectangularity, as follows:

\[
v_v = \beta_0 \cdot \frac{V_v}{b_v d} \quad \text{where } \beta_0 = \left( \frac{b_v}{b_0} \right)^{1/4} \tag{5}
\]

where \( b_v \) is the length of the longer side of the critical shear perimeter, and \( b_0 \) is the length of the shorter side of the critical shear perimeter. The shear strength ratio \( \nu/v_c \) according to the proposed equations is shown in Table 1. It can be seen that the inclusion of the ratio \( \beta_0 \) to the shear stress formula leads to a much improved prediction with a mean and coefficient of variation of 1.172 and 0.078, respectively.

SLAB-COLUMN CONNECTIONS TRANSFERRING UNBALANCED MOMENTS

ACI Unbalanced Moment-And-Shear Interaction

The ACI 318-05 equation for slab-column connections with unbalanced moment transfer can also be written in a form of an interaction equation, as follows:

\[
\frac{V_v}{V_o} + \frac{M_v}{M_o} = 1 \tag{6}
\]

where, \( V_v \) and \( M_v \) are the ultimate shear force and unbalanced moment at failure, respectively; \( V_o \) and \( M_o \) are the shear capacity in absence of unbalanced moment and moment capacity in absence of shear force, respectively, and they are expressed in Equation (7) and Equation (8) below:

\[
V_v = v_c b_v d \tag{7}
\]

\[
M_v = \frac{v_c J_v}{f'_c} \tag{8}
\]

where \( v_c \) is the ACI equations for shear strength determined from the lowest of Equations (1) to (3).

Proposed unbalanced moment-and-shear interaction

Equation (8) essentially defines the moment transfer strength as a function of connection geometry \( J \) and \( c \), shear strength \( v_c \), and the fraction of unbalanced moment transferred by shear \( v_c \). Note that the ACI equation for moment transfer strength shown in Equation (8), used to normalize the unbalanced moment, ignores the influence of flexural reinforcement. Moehle (1988) defines the moment transfer strength to be used in the unbalanced moment-and-shear interaction as the flexural capacity of slab reinforcement within a prescribed transfer width [3].

A linear interaction of shear and unbalanced moment is also used initially by the authors, assuming that the moment transfer strength \( M_f \) is limited solely by the available flexural reinforcement within an effective transfer width of \( c_f + 1.3h \) on each side of the column. The shear strength \( V_v \) is to be calculated by using the proposed equation for \( v_c \) given in Equation (4). A dimensionless parameter \( K \) is then introduced to relate the unbalanced moment-and-shear ratios. The basic format of the proposed unbalanced moment-and-shear interaction, applicable for both interior and exterior slab-column connections, is given below:

\[
\frac{V_v}{V_o} + K \frac{M_v}{M_f} = 1 \tag{9}
\]

<table>
<thead>
<tr>
<th>Column Geometry</th>
<th>Equation</th>
<th>Min ( v_v/v_c )</th>
<th>Max ( v_v/v_c )</th>
<th>Average ( v_v/v_c )</th>
<th>Standard Deviation</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Square columns</td>
<td>ACI 318-05</td>
<td>0.644</td>
<td>2.100</td>
<td>1.410</td>
<td>0.278</td>
<td>0.197</td>
</tr>
<tr>
<td>(223 specimens)</td>
<td>Proposed</td>
<td>0.858</td>
<td>1.691</td>
<td>1.232</td>
<td>0.169</td>
<td>0.137</td>
</tr>
<tr>
<td>Rectangular columns</td>
<td>ACI 318-05</td>
<td>1.033</td>
<td>1.570</td>
<td>1.267</td>
<td>0.171</td>
<td>0.135</td>
</tr>
<tr>
<td>(15 specimens)</td>
<td>Proposed</td>
<td>1.020</td>
<td>1.359</td>
<td>1.172</td>
<td>0.092</td>
<td>0.078</td>
</tr>
<tr>
<td>Total: 238 specimens</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The unbalanced moment-and-shear interaction of Equation (9) can be further improved significantly by raising the unbalanced moment ratio \( M_u / M_f \) to the power of \( 1/3 \). Therefore, the proposed unbalanced moment-and-shear interaction becomes:

\[
\frac{V_u}{V_c} + k \left( \frac{M_u}{M_f} \right)^{1/3} = 1
\]

(10)

where: \( V_u / V_c \) = gravity shear ratio; that is, the ratio of the ultimate shear force to the shear capacity based on Equation (4);
\( M_u / M_f \) = unbalanced moment ratio; that is, the ratio of the ultimate unbalanced moment to the moment transfer capacity calculated as the flexural capacity of slab reinforcement within the prescribed transfer width of \( c_f + 1.5h \) on each side of the column.

The Parameter K

The parameter \( K \) that relates shear and unbalanced moment ratio is very much dependent on the material properties and geometry of the connections. These factors include concrete compressive strength \( f'c \), flexural reinforcement ratio in relevant direction, slab effective depth \( d \), and slab critical shear perimeter \( b_c \). Further investigation on sets of test specimens from the literature with different levels of gravity loads showed that \( K \) also depends on the gravity shear ratio \( V_u / V_c \). Essentially, the influence of gravity shear ratio \( V_u / V_c \) on \( K \) has indirectly included the influences of \( f'c \) and \( \rho \) in the equation for \( V_u \), since \( f'c \) and \( \rho \) are used to calculate the shear strength \( V_c \) in Equation (4). Thus, an empirical equation for \( K \) that relates shear and unbalanced moment ratio is proposed here. The effects of gravity shear ratio and connection geometry are included as shown in Equation (11).

\[
K = \left(1 - 0.7 \frac{V_u}{V_c} \frac{d}{b_c} \right)^{0.1}
\]

(11)

Ultimate shear stress equation

The proposed unbalanced moment-and-shear interaction equation shown in Equation (10) can also be written in a form of an equation to calculate the ultimate shear stress at the slab critical shear perimeter. Assuming that shear failure occurs when the ultimate shear stress reaches the nominal shear strength, \( \nu_u = \nu_c \), then Equation (10) can be written as:

\[
\nu_u = \nu_c + k \left( \frac{M_u}{M_f} \right)^{1/3}
\]

(12)

Multiplying Equation (12) by \( \nu_c \), resulted in an equation to calculate the slab ultimate shear stress based on the proposed unbalanced moment-and-shear interaction as shown below:

\[
\nu_u = b_c d \frac{\nu_u}{b_c d} + k \left( \frac{M_u}{M_f} \right)^{1/3} \nu_c
\]

(13)

As discussed earlier in this paper, a factor \( \beta_e \) needs to be introduced into the shear stress calculation to consider column rectangularity; and thus, the complete equation to calculate the ultimate shear stress of slab-column connections becomes:

\[
\nu_u = \beta_e b_c d \frac{\nu_u}{b_c d} + k \left( \frac{M_u}{M_f} \right)^{1/3} \nu_c
\]

(14)

where: \( \nu_u, M_u = \) ultimate shear force and unbalanced moment at failure, respectively;
\( b_c, d = \) slab critical shear perimeter and effective depth, respectively;
\( \beta_e = \) column aspect ratio, (see Equation (5));
\( \nu_c = \) shear strength of slab, calculated using Equation (4);
\( K = \) a dimensionless parameter that relates shear and unbalanced moment ratio, calculated using Equation (11);
\( M_f = \) flexural capacity of slab reinforcement within the width of \( c_f + 1.5h \) on each side of the column.

The proposed equation for calculating the ultimate shear stress shown in Equation (14) is a general equation applicable for interior, edge and corner slab-column connections. For connections with biaxial moment transfer, Equation (14) should be applied independently for the shear and moment about each axis of the column, and the worst case controls the design.

EXPERIMENTAL VERIFICATIONS

The accuracy of the proposed formulas in Equation (4) for the pure punching strength and Equation (14) for the ultimate shear stress involving unbalanced moments had been verified against 455 slab-column connections data collected from the literature. Table 2 and Figure 1 summarize the performance of the proposed formulas in predicting the strength of interior, edge, and corner slab-column connections. Comparisons with the current ACI Code equations are also shown in Table 2. It can be seen from Table 2 that the mean values of the measured over predicted strengths \( \nu_u / \nu_c \), standard deviations and coefficients of variations from the proposed formulas are significantly better than those calculated based on the ACI equations.

CONCLUSIONS

A simple equation is proposed for calculating the slab ultimate shear stress at the critical shear perimeter of slab-column connections under gravity load and unbalanced moment transfer. The proposed method simplifies the design process and its accuracy and applicability has been checked against extensive experimental data. The performance of the proposed equations has been shown to be very good against 455 experimental connection specimens including interior, edge, and corner connections, ranging from symmetrical punching to connections having rectangular columns transferring unbalanced moments due to cyclic lateral loading.
Table 2. Summary of results for slab-column connections with moment transfer

<table>
<thead>
<tr>
<th>Connections</th>
<th>Equation</th>
<th>Min $v_u/v_c$</th>
<th>Max $v_u/v_c$</th>
<th>Average $v_u/v_c$</th>
<th>Standard deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior – sq column</td>
<td>ACI 318-05</td>
<td>0.724</td>
<td>2.011</td>
<td>1.202</td>
<td>0.257</td>
<td>0.214</td>
</tr>
<tr>
<td></td>
<td>Proposed</td>
<td>0.935</td>
<td>1.269</td>
<td>1.090</td>
<td>0.067</td>
<td>0.062</td>
</tr>
<tr>
<td>Interior – rec column</td>
<td>ACI 318-05</td>
<td>1.040</td>
<td>2.544</td>
<td>1.665</td>
<td>0.471</td>
<td>0.283</td>
</tr>
<tr>
<td></td>
<td>Proposed</td>
<td>0.949</td>
<td>1.494</td>
<td>1.181</td>
<td>0.119</td>
<td>0.101</td>
</tr>
<tr>
<td>Edge – sq column</td>
<td>ACI 318-05</td>
<td>0.807</td>
<td>2.527</td>
<td>1.413</td>
<td>0.382</td>
<td>0.271</td>
</tr>
<tr>
<td></td>
<td>Proposed</td>
<td>0.907</td>
<td>1.393</td>
<td>1.116</td>
<td>0.107</td>
<td>0.096</td>
</tr>
<tr>
<td>Edge – rec column</td>
<td>ACI 318-05</td>
<td>1.235</td>
<td>2.149</td>
<td>1.740</td>
<td>0.291</td>
<td>0.167</td>
</tr>
<tr>
<td></td>
<td>Proposed</td>
<td>0.915</td>
<td>1.381</td>
<td>1.196</td>
<td>0.123</td>
<td>0.103</td>
</tr>
<tr>
<td>Corner – sq column</td>
<td>ACI 318-05</td>
<td>1.061</td>
<td>6.490</td>
<td>1.96</td>
<td>1.053</td>
<td>0.479</td>
</tr>
<tr>
<td></td>
<td>Proposed</td>
<td>1.041</td>
<td>1.491</td>
<td>1.202</td>
<td>0.109</td>
<td>0.091</td>
</tr>
<tr>
<td>Corner – rec column</td>
<td>ACI 318-05</td>
<td>2.023</td>
<td>2.292</td>
<td>2.173</td>
<td>0.138</td>
<td>0.063</td>
</tr>
<tr>
<td></td>
<td>Proposed</td>
<td>1.291</td>
<td>1.411</td>
<td>1.332</td>
<td>0.068</td>
<td>0.051</td>
</tr>
</tbody>
</table>

Total: 217 specimens

REFERENCES

[1] ACI Committee 318, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary, American Concrete Institute, Farmington Hills, Mich., 2005, 443pp.


A Novel Numerical Method of Analyzing Elastic Solid Domain

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INTRODUCTION

Cellular Automata (CA) is a discrete model which consists of a regular grid of cells, each cell in one of a finite number of states. The development of CA can be traced back to 1940s when John Von Neuman et al. studied biological reproduction and crystal growth [1]. Up till today, CA has been successfully applied to various complex systems, such as theoretical biology [2], computability theory [3], Fluid dynamics [4] and traffic flow [5], etc.

This article first extends the concept of CA to 2-dimensional thermal conduction in solids, and then uses the same concept to simulate elastic plane stress problem. Numerical results show that the proposed approach has great potential in analyzing elastic solid domain.

BASIC CONCEPT

Consider an arbitrary 2-dimensional solid domain Ω shown in Fig. 1. The term \( v_i \) is a variable which has clear physics meaning at an arbitrary point \( P_i \) within Ω. In this article, the variable \( v_i \) can represent temperature or displacement. The point \( P_i \) is located at the center of its neighboring area which is illustrated by a circle with a radius \( r_i \). It is assumed that \( P_j \) is one of the monitoring points inside the neighboring area of point \( P_i \), and the number of such monitoring point is \( N_i \). Similar to \( P_i \), the physics variable at point \( P_j \) is denoted by \( v_j \).

Generally, a CA model is established on one or several specific rules. In this article, the basic rule for the proposed CA model is described as Eq. (1).

\[
v_i = \sum_{j=1}^{N_i} \omega_j v_j
\]

where \( \omega_j = \frac{1}{\sum_j [r_j - r_i]^\alpha} \) and the term \( \alpha \) is an exponential power to adjust the influence of the distance between points \( P_i \) and \( P_j \). In this article, \( \alpha \) is set to unity. It should be noted that if \( \alpha \) is set to zero, Eq. (1) becomes \( v_i = \frac{1}{N_i} \sum_{j=1}^{N_i} v_j \). The rule defined in Eq. (1) is based on the assumption that the variable \( v \) will not vary too much within a very small area, for instance, when radius \( r_i \rightarrow 0 \).

It should be noted that \( P_i \) is an arbitrary point within the domain \( \Omega \). One can define randomly many such points within the same domain and on the boundary of the domain, including point \( P_i \) itself. During the analysis, the value of the variable at each point is updated according to Eq. (1) until it converges to a stable value. The final values of the variable at each point form a state which represents the solution of the concerned domain.

NUMERICAL EXAMPLES

Example 1: Thermal conduction in a 2-dimensional domain

As shown in Fig. 2, the 2-dimensional domain is located in a \( xoy \) coordinate system. On vertical edges where \( x = 0 \) and \( x = 2a \), the temperatures are given as \( T = 500^\circ C \) and \( T = 100^\circ C \), respectively. On horizontal edges where \( y = 0 \) and \( y = a \), the boundary conditions are set to be adiabatic, i.e. no heat flow.

To solve such a classic problem, the proposed approach generates 1000 points within the domain and another 100
points on the boundary (Fig. 3). The temperature distribution as the central line \( y = a/2 \) is plotted in Fig. 4. One finds that the numerical result is accurate as compared with the analytical solution.

Example 2: Deformation of a plate
Consider a square plate. The boundary conditions are shown in Fig. 5. In the figure, the terms \( X_x \) and \( X_y \) represent the tractions in \( x \) and \( y \) directions, respectively, while \( U_x \) and \( U_y \) are the displacements. \( U_{x,y} \) denotes the derivative of displacement \( U_i \) with respect to \( y \). From Fig. 5, it can be seen that on the edges where \( x = 0 \) and \( x = 1 \), the tractions are all zero. In this case, Eq. (1) cannot be directly used. However, the traction boundary conditions can be expressed by the derivatives of displacements with respect to coordinates through force equilibrium:

\[
\left[ G(U_{i,j} + U_{j,i}) + \lambda U_{i,j} \delta_{i,j} \right] \eta_j = X_i \quad (i,j = 1,2) \tag{2}
\]

In Eq. (2), direction ‘1’ means the \( x \) direction while the direction ‘2’ means the \( y \) direction. The term \( G \) is the shear modulus of the plate material and \( \lambda \) is the Lame constant.

Eq. (2) relates the boundary tractions to the derivatives of the displacements with respect to coordinates. Hence, if the derivatives of the displacements are replaced by the differences, the rule given in Eq. (1) can be applied. In this example, 100 points are generated on the four edges of the plate, while inside the domain, 600 points are randomly distributed.

The results are demonstrated in Fig. 6. In the figure, the red dots form the geometry of the original plate, and the black dots outline the deformed shape of the plate. It can be seen that the deformation of the plate is very realistic.

CONCLUSIONS
This article extends the concept of cellular automata to 2-dimensional elastic domain. The novelty of the proposed approach is that, this approach is able to simulate the response of a complex problem using a very simple concept. Unlike the FEM and BEM approaches, the proposed approach is a completely mesh-free method. It is also different from the existing meshless methods since the CA approach does not form any stiffness matrix and therefore does not solve matrix equation.
Another advantage of the proposed approach is that it can be easily extended to 3-dimensional problems. The authors expect that more attention will be paid to the relevant research in time to come.

REFERENCES


Experimental Investigation of Steel Beam-to-Column Joints at Elevated Temperatures

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INTRODUCTION

In conventional steel structural analysis at ambient temperature, the beam-to-column joint is defined as either “pinned” (no moment resistance) or “rigid” (full moment resistance). However, the actual joint behaviour lies somewhere between these two extremes called as a “semi-rigid” joint. Until recently, the provision of adequate fire protection for steel and composite structures is mainly through prescriptive path, that is, by applying sufficient fire protection materials for beams and columns to keep their temperatures from growing to limiting temperature values. This represents a significant addition to the construction cost, and places steel structures at a disadvantage compared with concrete structures. Therefore, minimizing the cost of fire protection has provided a strong impetus to research work on fire effects on steel structures and components. Some research works have been conducted to investigate the influence of temperature on steel structures and isolated members. Because steel members are assembled together through joints, there is a pressing need to better understand the joint behaviour and its effect on overall behaviour of steel framed structures under fire conditions. However, to-date, there has only been a limited number of experimental investigations conducted with fairly limited types of joints at elevated temperatures by Lawson (1990), Leston-Jones et al. (1997), Al-Jabri (1998), Spyrou et al. (2002), Vimonials et al. (2006), etc. The study described here focuses on establishing elevated temperature rotational characteristics for an extended end-plate beam-to-column steel joint in the presence of axial compression.

TEST ARRANGEMENT AND SETUP

In total, there were six extended end-plate beam-to-column joints tested as “cruciform” assemblies in two groups. A typical cruciform specimen, consisting of two 1.6 m UB 305x102x25 kg/m beams symmetrically framing into the flanges of a 1.5 m high UC 254x254x107 kg/m section, as shown in Fig. 1. In the first group, three cruciform specimens (CR1, CR2 and CR3) were tested at 700 °C with axial compressive force maintained at 0, 2.5 and 4% of the plastic squashing capacity at ambient temperature. In the second group, three more cruciforms (CR4, CR5 and CR6) were tested at 400, 550 and 700 °C, respectively, this time without thermal restraint.

Test Set-Up

The cruciform testing system (Fig. 2) consisted of an enclosed electrical furnace (o), a vertical hydraulic jack (n) and an axial
restraint system (j,p,s). The hydraulic jack (n) was utilized as a loading device attached to an external portal frame system (m). A horizontally-mounted hydraulic jack (j) provided axial force onto the beam sections of the cruciform specimen. In addition, there were two forked supporting systems (q) to prevent the out-of-plane rotations of beam ends.

Loading Facility and Restraint Systems
The whole loading system consisted of a hydraulic jack (n) to provide vertical load at the top of cruciform specimens, and the two-portal-frame reaction system (p), as shown in Fig. 2. Another axial restraint system was designed and fabricated to provide the prescribed axial restraint force onto the beam sections of the cruciform specimen.

Deflection Acquisition System (LVDTs)
To obtain a good description of the development of in-plane deflections, ten sets of linear variable differential transducers (LVDTs as L2 to L11) were utilized to monitor the beam vertical deflections for CR1 to CR3. For the unrestrained cruciform tests (CR4 to CR6), in order to obtain explicit rotation measurements of the beam sections, two sets of LVDTs (D9 to D12) were placed at each end of the beam section outside the furnace at ±120 mm distance from the horizontal central axis.

RESTRAINED CRUCIFORM TEST DEFLECTION CHARACTERISTICS
From the vertical beam deflection measurements in Fig. 3, negative deflection measurements can be found for both left and right beams prior to loading. This hogging-up movement was due to axial compressive force and to thermal bowing of the beams at elevated temperature. It can also be seen that the beam shows linear load-deflection characteristics before the load reaches 30 kN. When the external load reaches 40 kN, the beams start to sag downwards. Applied compression can induce a significant increase of beam deflections due to P-δ effect.

UNRESTRAINED CRUCIFORM TEST DEFLECTION CHARACTERISTICS
Typical specimen CR 5 was loaded up to failure at 550°C. Beam vertical deflections recorded by transducers are shown in Fig. 4. Initial negative deflections can be seen for both the left and the right beams due to the higher test temperature. Beams show a smooth deflection response until the vertical load reaches 150 kN. Above this limit, there is a rapid increase in vertical deflection, causing a plateau in the load-deflection relationships, indicating run-away failure of the joints.

CRUCIFORM TEST FAILURE MODES AND SUMMARY
The first group of specimens (CR1 to CR3) was tested at 700°C, with axial load ratios of 0, 2.5% and 4% of the beam’s plastic squash capacity. In addition to out-of-plane deflection
of the beam web panels, significant bending deformation can be observed at the end-plate adjacent to the beam bottom flange (in tension), as shown in Fig. 4. Clearly CR1 shows the greatest end-plate bending deflection, due to maximum tensile force transferred from the beam bottom flange.

Fig. 5  Failure Modes for CR4

For the second group of cruciform specimens, the test results show that both the left- and the right-hand beam web panels developed out-of-plane deflections adjacent to the end-plates. Fig. 5 shows the experimental failure modes for CR4 specimen tested at 400°C. It can be seen that the beam web panels experienced tension field action similar to shear failure mode in a plate girder web panel (Vimonsatit et al. 2006).

A typical overall failure mode is shown for CR4 in Fig. 6. This specimen is selected as it represents a typical behaviour of a cruciform specimen under fire conditions. It also has a more uniform temperature field than the first three specimens and its behaviour was quite similar to that of CR5 and CR6. It can be seen that tensile yield zones are formed in the beam web panels adjacent to the column flanges. Out-of-plane deformations of the left- and right-hand beam webs are in opposite directions, the anti-symmetry being possibly induced by small rotations of the column. Clearly, under fire conditions, shear deformation is significant even in universal rolled sections and should be taken into account in joint modelling when using a “Component-based” method.

CONCLUSIONS

In this research, six steel cruciform assemblies have been tested at elevated temperatures, with and without thermal restraint effects. Moment-rotation-temperature characteristics of both the restrained and unrestrained steel beam-to-column joints have been obtained. It is anticipated that these results will be beneficial in validating “Component-Based” modelling for steel joints under fire conditions. Such models offer the most practical and economical way of introducing a predictive capability into whole-frame modelling in fire, since it is impractical for designers to use either complex finite element analysis or expensive furnace testing to model joint behaviour.

REFERENCES


INTRODUCTION

In Singapore, there is an increasing need to plan and develop strategic underground facilities and infrastructures for various military and non-military applications in recent years. The lack of adequate and accurate 3D subsurface geological structures often makes the design and construction of such underground works difficult. 3D geological modelling and visualization, using Geographical Information System (GIS), could help to “visualize” the underground space. The resulting 3D Geological Information System (3DGIS) could effectively store, display and analyze a variety of subsurface data through an integrated database.

Prototype of a 3DGIS called 3DRock was developed by the authors by extending the functionalities of “VGEGIS” – a “Virtual Geographic Environment” GIS software developed using VC++ programming language by the State Key Laboratory of Information Engineering in Surveying, Mapping and Remote Sensing (LIESMARS) of Wuhan University, P.R. China. The functions of the VGEGIS package enable one to visualize, explore and interact with the 3D data of a “CyberCity”. The proposed extension will enable 3DRock to deal with the aboveground, surface and subsurface information.

PROTOTYPE OF 3D GEOLOGICAL INFORMATION SYSTEM

3DRock comprises five modules, i.e. GeoDatabase, 2D View & Analysis, 3D Modelling, GeoAnalysis and GeoVisualization. The relationships between the various modules are illustrated in Figure 1.

GeoDatabase provides an integrated database management system for all the spatial data sets that represent the aboveground, surface and subsurface information. Subsurface data include borehole data, seismic data, cross section, geological map etc. The borehole data will comply with the AGS (Association of Geotechnical and Geoenvironmental Specialists U.K.) format. Seismic data, cross section and geological map are to be digitalized in DXF format and converted as borehole data for modelling.

3D Modelling module models all the objects in the real world. Multi-DEM algorithm is used to construct layer-based geology. Modelling complex geological objects in 3D is a challenge and still being studied.

GeoVisualization provides 3D perspective views that show features and feature relationships above or under the ground. GeoAnalysis module, for subsurface data, supports geological analysis functions.

Extension of VGEGIS to model subsurface objects, which are diametrically different from the aboveground ones, requires the resolution of a few critical issues in 3DRock:

- Extension of 3D data models of VGEGIS package to facilitate the visualization and GIS operations of subsurface features;
- Support for integration of vast amounts of data with different sources and resolutions, such as borehole data, seismic data and cross section data etc.;
- Modelling of complex geological objects, such as fault, fold and unconformities etc.;
- Support for geological analysis functions, such as making cross sections, fence diagrams, isopach maps etc.

DATA MODELS OF 3DRock

A unified 3D data model, given in Figure 2, is proposed to handle continuous and discrete feature objects both on the ground and under the ground. In Figure 2, the data models enclosed by the blue rectangle are designed for subsurface modelling. The rest is the data models of VGEGIS. The triangle-shaped symbol denotes an inheritance relationship.

Borehole is a unique feature in subsurface modelling - there is no similar object aboveground. It comprises one or more cylinder primitives.
Faults are very important for studying a geological body. A fault is a surface or narrow zone across which there has been relative displacement of the two sides parallel to the zone. It will be modelled as surface with some special attributes.

The volume object like buildings is dealt as surface models in VEGIS. In 3D Rock, facial model will also be used to model geological strata with boundary representation (BR) technique.

The data structure of 3D object is one of the most significant factors for efficient data access and fast rendering. The structures of borehole and fault plane should be designed carefully.

GEOLOGICAL MODELLING WITH MULTIPLE-DEM

3D geological modelling is based on multiple-DEM with boundary representation (BR) technique. Each stratum model is represented by the top and bottom DEMs and side faces.

The multiple-DEM algorithm is based on the assumption that geological stratum is in a depositional sequence. Simply stated, all geologic strata throughout the domain are ordered from bottom to top and a consistent hierarchy is used for all borelogs.

The benefits to this type of display and modelling include the ability to turn individual surface on or off and the ability to edit individual surfaces. Figure 3 shows the preliminary result of a geological modelling.

CONCLUSIONS AND RECOMMENDATIONS

One prototype of 3D Geological Information System and some key issues, like data models and geological modelling, have been briefly presented. However, modelling complex geological objects is a challenging area to the study and more efforts are required to implement the spatial analysis.
Automated 3D Geological Surface Modelling With CDTIN

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Zhang Xianhui (zh0002ui@ntu.edu.sg)

INTRODUCTION

Advances in information technologies and Geographic Information Systems (GIS) make it possible to create 3D and interactive geological information system. Traditional 2D geological maps in DXF file format depicting geological structures and features, provide vast amount of source data. Using VGEGIS software of Wuhan University, a fast and efficient method was developed to automate the generation of 3D geological surface using Constrained Delaunay Triangulation (CDT).

Prototype of a 3D Geological Information System, called 3DRock is proposed in this study by extending the VGEGIS software which integrates both vector and raster data. Vector data represent geological features with points, lines and polygons. Raster data represent geographic features using a pattern of discrete cells called pixels, such as satellite images, aerial photographs and a variety of scanned images.

DATA TRANSFORMATION

The basic element of a 3D geological map is the Digital Elevation Model (DEM) which is a digital raster representation of the 3D terrain surface. Each geological feature is represented as vector data in a 2D digital map. Each polygon defines the boundary of a geological object and its conversion to 3D surface requires the construction of the CDT covering the polygon. The CDT ensures that the edges of the polygons are also the edges of the triangles forming the triangulated irregular network (TIN). The steps of data transformation include: (1) search all vertices of the DEM inside the polygon and construct the CDT; (2) trim triangles outside the polygon and (3) texture/material mapping of the CDT surfaces. Figures 1 to 3 illustrate these procedures.

DEM SIMPLIFICATION

The size of a densely sampled DEM data can easily exceed the capabilities of typical graphics hardware and thus makes interactive application inefficient. Level of Details (LOD) algorithm enables the viewer to see a small portion of the whole data set at each viewpoint. Scenes farther away from the viewer need not be rendered in the same detail as those nearer the viewer. The algorithm used in this study employs regular grids managed in a binary tree data structure and it divides a large terrain into smaller blocks. A polygon with 6030 line segments was re-triangulated to reduce the number of triangles by ignoring intermediate grid point, i.e. in between two grid points, with height difference of less than 0.0 m (original resolution), 0.5 m, 1.0 m, 1.5 m, 2.0 m and 2.5 m. This will simplify the DEM. Table 1 illustrates the effect on the processing time with respect to the different resolutions, the number of DEM vertices and the number of triangles. It is apparent that there is about a 12-fold saving of processing time by ignoring height differences of 2.5 m. Figures 4 and 5 illustrate the result CDT with 0.5 m and 2.5 m height simplifications.
CONCLUSIONS AND RECOMMENDATIONS

Using CDT and simplification of DEM, this paper illustrates an efficient and effective method of creating 3D geological surface from 2D geological map.

Table 1: The result of LOD simplification

<table>
<thead>
<tr>
<th>Resolution</th>
<th>Point</th>
<th>Triangle</th>
<th>Time(s)</th>
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<tr>
<td>0.0 m</td>
<td>90000</td>
<td>46192</td>
<td>97.260</td>
</tr>
<tr>
<td>0.5 m</td>
<td>10041</td>
<td>9977</td>
<td>15.640</td>
</tr>
<tr>
<td>1.0 m</td>
<td>4529</td>
<td>7523</td>
<td>11.094</td>
</tr>
<tr>
<td>1.5 m</td>
<td>2880</td>
<td>6742</td>
<td>9.313</td>
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<td>2.0 m</td>
<td>2199</td>
<td>6479</td>
<td>8.172</td>
</tr>
<tr>
<td>2.5 m</td>
<td>1879</td>
<td>6337</td>
<td>7.844</td>
</tr>
</tbody>
</table>

Figure 3: Drape the result surface with red texture onto the DEM (in grey)

Figure 4: The result CDT after DEM simplification with 0.5 m grid

Figure 5: The result CDT after DEM simplification with 2.5 m grid
INTRODUCTION

The rapid economic growth in Singapore over the past three decades has raised the standard of living considerably, resulting in a greater desire for people to own and use cars. Per capita car ownership has risen from 0.059 in 1976 to 0.101 in 2005, or an increase of 71.2%. Together with population growth, this means there are now many more cars demanding the use of road space. Since Singapore is a small country, there is limited scope for more road building without severely disrupting the life and environment of the city. Economic and social costs would also be incurred. This has led to a mixture of usage and ownership restraint measures in Singapore over the years.

The usage restraints include the Area Licensing Scheme which was introduced in 1975 and replaced in 1998 by the Electronic Road Pricing scheme, and high petrol taxes. Ownership restraints consist of vehicle taxes and registration fees as well as the Vehicle Quota System (VQS), more commonly known as the certificate of entitlement (COE) scheme.

Restraint on ownership is based on the rationale that vehicle owners tend to make more trips and thus contribute to congestion. There is evidence from surveys conducted in Singapore that vehicle-owning households make more trips. Many cities have adopted high vehicle taxes to subdue ownership demand. Singapore is the only country with direct control in the growth of vehicle population.

VQS was introduced in May 1990. The system applies to all vehicles except emergency vehicles, public and school buses, diplomatic vehicles and vehicles for the disabled. Under VQS, anyone who wishes to own a vehicle must have a COE. Based on the traffic which the road infrastructure can cope with, the authorities have set the annual quota at 3% of the vehicle population (subject to annual review) plus “replacements” for vehicles that have been de-registered. The quota is administered through the release of COEs for tendering by the public.

This study investigates the desire to own cars in Singapore and the effectiveness and consequences of VQS in controlling car ownership.

DESIRE TO OWN CARS

A number of factors are likely to affect people’s desire to own cars in Singapore. For example, economic conditions, income level, car and fuel prices and population or number of households. To investigate the underlying desire for car ownership, an analysis of data before 1990 (i.e., before VQS was implemented) was conducted. This analysis showed a strong correlation between car population and economic conditions represented by Gross Domestic Product (GDP) in Singapore.

Two statistical relationships between car population (CAR in thousands) and GDP (in S$ millions at 2000 prices) were studied using data extracted from information published by the Land Transport Authority and the Department of Statistics. The first statistical model relates car population in a year with the GDP for that year. The other relates car population in a year to the GDP of the preceding year as there may be a time lag between the decision in car buying and economic conditions.

The resulting models are presented below.

\[ CAR = 37.983 + 0.00325 \text{GDP} \quad (R^2 = 0.95) \quad (1) \]

\[ (3.67) \quad (14.9) \]

\[ CAR = 34.936 + 0.00357 \text{LGDP} \quad (R^2 = 0.98) \quad (2) \]

\[ (5.02) \quad (22.7) \]

In the above models, the dependent variable “Car” includes all except tax-exempted cars and station-wagons. The variable “LGDP” represents GDP lagged by one year. The numbers in parentheses give the t-statistics associated with the constant term and the explanatory variable (GDP or LGDP), which are all significant. Both models have high R-squared values of 0.95 and 0.98, indicating they provide strong fits to historical car population levels, and the model with GDP lagged by one year gave a slightly better fit. This can also be seen from Figure 1 which presents a plot of the actual and predicted car population levels between 1976 and 1989.

EFFECT OF THE VEHICLE QUOTA SYSTEM

The car population in Singapore grew from 134,233 at end-1976 to 258,537 at end-1989, an increase of 92.6% over the 13 years before VQS was introduced in 1990. Over the next 13-
year period from end-1990 to end-2003, car population grew from 269,436 to 405,328, or an increase of 50.4%, which is only 54% of the increase from 1976 to 1989. This demonstrates the effectiveness of the VQS in controlling car population growth in Singapore. It also implies that there is significant unsatisfied demand for car ownership.

To estimate this restraining effect of VQS on car ownership, the model given in Eq. (2) is used to estimate car ownership levels that would likely be there if VQS had not been implemented. The results of this estimation are plotted in Figure 2 together with actual car population for the years after the implementation of VQS. The difference between the curves for any given year would likely represent the unsatisfied demand for car ownership in that year due to the VQS. It is noted here that there is a limit on how high unconstrained demand for car ownership can rise. Therefore, the use of the model presented here in estimating unsatisfied demand needs to be used with caution.

An obvious question of interest resulting from this analysis is the effect of unsatisfied car ownership demand on the number and amount of bids submitted in tendering for the Certificates of Entitlement. However, investigation into this is rather complicated due to repeat bids by potential car buyers as well as people who submit very low bids in the hope of getting lucky in tendering. The rule that all successful bids pay the same amount (lowest successful bid) also complicates the matter. Hence, this issue remains to be studied in further research.

**Behaviour of Elderly Pedestrians at Signalised Junctions in Singapore**

Lum Kit Meng (ckmlum@ntu.edu.sg)

**INTRODUCTION**

Elderly pedestrians are more prone to accidents and are more likely to be severely injured or killed if they are involved in an accident as compared to their younger counterparts. There is also considerable evidence to show that the elderly experience shortfalls not only in their physical abilities but also in sensory, perceptual, and cognitive abilities. Little is known, however, about how these would affect road crossing behaviour and their safety, and the extent to which age-related mobility, sensory or motor deficits would affect their road crossing ability and whether they are able to adopt compensatory behaviour to overcome these disabilities. The purpose of this study is therefore to investigate the behaviour of the elderly pedestrians at signalised junctions and to determine whether the pedestrian clearance timing is sufficient.

**DESIGN OF THE PEDESTRIAN CLEARANCE TIMING**

The Highway Capacity Manual (HCM) 1994 initially cited 1.35 m/s as the typical walking speed in a crosswalk but 1.2 m/s as the assumed 15th percentile crosswalk walking speed when pedestrian timing requirements are computed. Subsequently, HCM 2000 recommends an average walking speed of 1.2 m/s where there are 20% or less elderly pedestrians and 1.0 m/s where there are greater than 20% elderly pedestrians. As a result, the 1.2 m/s “normal” or average walking speed gradually evolving into a 1.2 m/s design speed for the pedestrian clearance interval, with slower speeds suggested only for special situations in which an unspecified number of elderly pedestrians are expected to cross. Although the authorities are generally aware of the need to provide sufficient pedestrian crossing facilities to meet demand and safety requirements, it was found that the attendant effect of the elderly pedestrians on the pedestrian clearance interval has not been sufficiently addressed in the design standards.

The pedestrian clearance (T) or alternately known as the “Green Man” timing is a function of road crossing distance (d), walking speed (v) and the start-up timing (s). Mathematically, T can be expressed as the road crossing distance (d) divide by the walking speed (v) plus the start-up timing (s). The start-up timing is defined as the time interval required by sensitive pedestrians seeking to confirm the traffic situation and that the gap presented is indeed adequate for their needs. In Singapore, a walking speed of 1.2 m/s and a start-up timing of 5 seconds are typically adopted in the design of the pedestrian clearance timing.
STUDY METHODOLOGY

The behaviour of the elderly pedestrians at signalised junctions can be gathered by a perception survey and field observational studies. The major concern regarding the usage of perception survey alone is that the findings obtained may not necessarily reflect the actual crossing behaviour of the elderly pedestrians. On the other hand, the sole purpose of field observational study which provides an insight into pedestrians’ actual crossing behaviour is not without its limitations, that is, not all scenarios can be captured as compared to a controlled environment. This research study thus focused on a combinational approach using both methods of data collection to provide a complementary analysis of pedestrian behaviour at signalised junctions.

PERCEPTION SURVEY FINDINGS

The main issues studied in the perception survey include, among many others, pedestrian behaviour and the adequacy of the “Green Man” timing. Most of the survey questions were constructed on a two level basis. The top level requires their views on specific issues while the lower level deals with the reasons as to why they had chosen such responses. A total of 256 respondents were randomly chosen for the main perception survey, of which 119 of them were elderly pedestrians defined as those 65 years old and above. Three in four elderly respondents expressed their preference for crossing at signalised junctions over other types of pedestrian crossing facilities. Overhead bridges and underpasses were considered to be “troublesome”, “inconvenient” and “not user-friendly” by most of the elderly respondents.

One in two respondents would consider crossing the road if they reached the junction during the display of the steady “Green Man”. Reasons for choosing to cross include long waiting time for the next cycle, being rushed for time, absence of vehicles on the road and the signal still on the “Green Man”. On the other hand, upon reaching the signalised junction during the flashing of the “Green Man”, one in five respondents would choose to cross. However, a high proportion of the elderly preferred to wait for the next “Green Man” as compared to their younger counterparts (see Figure 1).

Four in five respondents had no difficulty in completing the crossing within the stipulated crossing timing. The remaining respondents (about 20%) were mainly elderly pedestrians. The difficulties they experienced are due to problems with negotiating curbs, relying on walking aids and the presence of too many pedestrians crossing at the same time. However, seven in ten respondents thought that the “Green Man” timing should be increased. Some reasons cited include that elderly pedestrians generally walked at a slower pace, and that a shorter timing would make them feel anxious when crossing. This psychological barrier would further impede their walking speed as they become more hesitant during crossing. On the other hand, this feeling of anxiety might make them more cautious.

FIELD OBSERVATIONAL FINDINGS

From the field observational data gathered, a number of analyses were carried out. Of these, the start-up timing and the walking speed of pedestrians crossing at signalised junctions are presented. On-site observations revealed that the start-up timing was significantly affected by the physical environmental factors rather than by the individual pedestrian behaviour. These physical factors include the width of the crossing, the size of the pedestrian waiting area and the number of pedestrians crossing at each interval. A high volume of pedestrian traffic and a narrow walking width would create a bottleneck which would translate into a longer start-up timing as not all the pedestrians were able to step onto the road to commence their crossing. Nevertheless, the findings revealed that the pedestrian start-up timing of 5 seconds was generally sufficient for most waiting pedestrians to initiate their crossing.

A total of 1188 pedestrians were observed crossing the junction during the display of the steady “Green Man” at the four study sites. Of which, 967 of them were observed and classified as elderly pedestrians. The mean walking speeds for various categories of pedestrians at the four study sites are tabulated in Table 1. In general, the walking speeds of the younger road

<table>
<thead>
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<th>Sites</th>
<th>Junction Approach 1</th>
<th>Junction Approach 2</th>
</tr>
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<tbody>
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<td></td>
<td>Young Male</td>
<td>Old Male</td>
</tr>
<tr>
<td>A</td>
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</tr>
<tr>
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</tr>
<tr>
<td>C</td>
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<td>1.25</td>
</tr>
<tr>
<td>D</td>
<td>1.33</td>
<td>1.26</td>
</tr>
</tbody>
</table>

Figure 1. Crossing the road during the flashing of the “Green Man”
users were much faster than the elderly as the former were quicker and more agile. The elderly usually experience impeded movements due to factors such as walking disability that requires the use of quad-pone cane, walker or crutches, carrying or pushing of heavy loads or assisting children to cross. The overall average walking speed for the elderly is calculated to be 1.23 m/s. This is very close to the design speed of about 1.2 m/s. However, it was significantly higher than the 15th percentile speed of 1.07 m/s for the elderly pedestrians.

CONCLUDING REMARKS

With the context of the experimental design, the start-up timing of 5 seconds was generally sufficient for most waiting pedestrians to initiate their crossing. The design walking speed of 1.2 m/s was also sufficient for all young pedestrians and a majority of the older pedestrian to complete their crossing. However, if there are many elderly pedestrians, it would be more appropriate to use the 15th percentile walking speed instead. This is to ensure that they are able to safely complete their crossing within the stipulated timing.

Advance Warning Devices at Signalised traffic Controls

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An Advance Warning Light (AWL) is a device that is used to convey the status of traffic signal indication at a signal-controlled site. It is basically a set of traffic lights positioned some distance before a signalised road junction or a signalised mid-block pedestrian crossing. The AWL, when activated, serves to provide advance indication to the motorists that the signal lights ahead will be turning amber or be against their favour during their approach.

Singapore implemented its first AWL (along Rochor Road) in 2000. By the end of 2004, the scheme has been extended to 32 locations which included 14 signalised mid-block pedestrian crossings. Overseas experience with AWL operations has shown that the AWL, when installed at strategic locations, can result in better road safety performance.

The impact of AWL on road safety performance was studied that entailed an examination of road traffic accidents at the AWL sites in Singapore. At the point of study, 24 of the 32 AWL sites had AWL in operation for between 16 to 91 months (excluding a 3-month period at implementation), with a median duration of 31 months. These 24 sites were chosen such that accident counts for equal before and after durations at each site were assembled, and the counts for the 24 sites were then aggregated for before-after comparison. Only accidents with human casualties (fatality and/or injury) were included in the analysis.

Accidents at the 24 AWL sites were found to be generally quite few, with a greater occurrence rate at signalised mid-block crossings and signalised T-junctions than signalised cross-junctions (see Figure 1). In terms of severity, most accidents entailed only slight injuries (see Figure 2). The majority of the accidents were collisions between moving vehicles (see Figure 3), of which the head-side collisions were the most common (see Figure 4). Comparison of accident counts in the before and after periods revealed tangible reductions in accident occurrences, especially at the T-junction AWL sites. There was a large drop in the predominant head-side collisions, and this improvement is significant given that the head-side collision is the most costly accident type, and it is also the most disruptive to traffic operations. The findings from this study show that the implementation of AWL can enhance safety performance. The present results are based on a relatively small sample, and investigations are continuing to establish more definitive findings which shall use a larger sample size and a longer data series as they become available over time.

Figure 1. Before and after accidents by location type

Figure 2. Before and after accidents by severity
Figure 3. Before and after accidents by nature of collision

Figure 4. Before and after accidents by type of collision between vehicles
Power-law Index for Velocity Profiles in Open Channel Flows

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INTRODUCTION

Practical applications in hydraulic engineering have demonstrated that velocity profiles measured in wide open channels can be well represented by the power law. However, the selection of the power law exponent or index appears quite empirical and even subjective. In spite of some theoretical efforts made recently, the power law is usually considered as the simple data-driven formulation, in comparison with the log law that can be theoretically derived based on dimensional reasoning or theoretical arguments.

On the other hand, the evaluation of the same exponent is also essential for formulating flow resistance in the form of power function since the flow resistance equation can be derived from the average velocity. For example, the well-known Manning equation, when scaled with the shear velocity, gives the selection of the power law exponent or index appears quite empirical and even subjective. In spite of some theoretical efforts made recently, the power law is usually considered as the simple data-driven formulation, in comparison with the log law that can be theoretically derived based on dimensional reasoning or theoretical arguments.

This study aimed to present certain theoretical grounds for the derivation of the power law. Its connection with the log law is explored in light of improved understanding of wall turbulence developed in the past few years. This paper details how the power-law index varies with flow and boundary conditions, and what approximations could be made for practical applications.

ANALYTICAL CONSIDERATIONS

Turbulence theory has shown that the log law applies only for the overlap region. The analysis to be carried out in this study will be first limited to this region so that the log law can be engaged as a theoretical basis. For uniform flows in a wide open channel, the power law can be expressed as

\[ \frac{u}{u_m} = \left( \frac{y}{\kappa} \right)^{-m} \]

(1)

where \( u \) is the streamwise, time-mean flow velocity, \( u_m \) is the maximum flow velocity measured at the free surface (\( y = h \)), \( y \) is the bed-normal distance measured upwards from the channel bed datum, \( h \) is the flow depth, and \( 1/m \) is referred to as the power-law exponent or index. For the overlap region, the log law is

\[ \frac{u}{u_*} = \ln \left( \frac{y}{y_o} \right) \]

(2)

where \( y_o \) is the location of the zero-velocity predicted by the log law and also called hydrodynamic roughness length.

The upper limit of the overlap layer is the same as that of the inner region. Here we denote it by \( y = y_o \), and the corresponding velocity by \( u = u_* \). With Eq. (2),

\[ \frac{u}{u_*} = \frac{1}{k} \ln \left( \frac{y_o}{y} \right) \]

(3)

Using Eqs. (2) and (3) to eliminate \( u \), one gets

\[ \frac{u_*}{u} = y_o^\alpha y^m \]

(4)

where \( \alpha = \ln(y_o/y) \), \( u_* = ul_u \), and \( \eta = y/y_o \).

Similarly, the power law is expressed as

\[ \frac{u_*}{u} = \eta^\phi \]

(5)

To compare the two functions given by Eqs. (4) and (5), two conditions are imposed. \( \phi = \phi_o \), and \( d\phi_o/d\eta = d\phi/d\eta \), which yields that the power law index is given by

\[ m = \ln \left( \frac{\phi}{\phi_o} \right) \]

(6)

To evaluate this index, it is necessary to first understand how \( y_o \) and \( y_* \) are associated with flow and boundary conditions.

(a) Hydrodynamic roughness length, \( y_o \)

The hydrodynamic roughness length is generally dependent on two typical length scales, the viscous length scale defined by \( \nu/u \), and the size of the channel bed roughness that could be described by the diameter of bed sediment particles. Such dependence can be quantified using classical Nikuradze’s pipe flow data, from which an empirical formula is derived here (see Fig. 1),

\[ y_o = 0.11 \frac{y}{u} + 0.03k + 0.115 \exp \left( -0.04 \frac{2k}{\nu} \right) \]

(7)

It should be mentioned that the interpolation proposed previously by Colebrook for pipe flow resistance is simply a linear combination of the scales, i.e. \( y_o = 0.115k + 0.03\nu/u \), which fails to fit the Nikuradze’s data for the transitional bed condition.

(b) Thickness of inner region, \( y_* \)

Empirically, the thickness of the inner region is taken as 20% of the flow depth for wide open channels, i.e. \( y_o/h = 0.2 \). Physically, the thickness can be considered as a measure of the extent to which the boundary effect interacts with the flow inertia. Such interactions can be further characterized by turbulence statistics, for example, the location of maximum velocity fluctuation or shear stress. Based on recent studies in this respect, we here assume that for smooth open channel flows,
\[ \frac{y_o}{h} = \zeta \left( \frac{Uh}{v} \right)^{1/3} \]  

where \( \zeta \) and \( \lambda \) are constants. With the relation of \( y_o = 0.11v/u^* \) and Eq. (8), it can be shown that for the case of smooth boundary \( \lambda = -0.208 \) and \( \zeta = 1.661 \), and thus

\[ \frac{y_o}{h} = 1.834 \left( \frac{h}{y_o} \right)^{1/3} \]  

(9)

Eq. (9) shows that the location of the upper edge of the overlap layer scales both with the flow depth and hydrodynamic roughness length. It could be expected that similar scaling relation may also exist for rough boundary conditions. However there is no information available at the current stage for understanding effects of bed roughness on the logarithmic overlap region. Therefore, as a first approximation, Eq. (9) is assumed also applicable to all flow regimes in this study.

Using Eqs. (6), (7) and (9), the variation of \( m \) with the Reynolds number and also the relative roughness height is plotted in Fig. 2. It is interesting to note that the average \( m \)-value for the condition of Nikuradze’s experiments can be roughly taken to be 6, which gives the same index included in the Manning equation. This implies that the one-sixth power could serve as an acceptable approximation for many flow conditions, which are subject to different boundary roughness and Reynolds numbers.

Fig. 1 Variations of hydrodynamic roughness length for different flow regimes.

Fig. 2 Dependence of the power-law index on Reynolds number and relative roughness height.

Fig. 2 Dependence of the power-law index on Reynolds number and relative roughness height.

For \( u^*/U \) or the friction factor involved in the above computation, the following formula is proposed here based on Nikuradze’s data (Fig. 3),

\[ f = \left[ 0.16 \left( \frac{4h^2}{y_o^2} \right)^{1/3} \left( 2 \log \left( \frac{2h}{k_s} \right) - 1.74 \right)^{-1/2} \right] \]  

(10)

where \( Z = 1/\left[ \left( 4h^2/y_o^2 \right)^{1/3} \left( 2 \log \left( 2h/k_s \right) - 1.74 \right)^{1/2} \right] \).

In addition, since both \( m \) and \( f \) depend on \( Uh/V \) and \( h/k_s \), \( m \) could be also computed based on \( f \) only. This possibility is also explored here, showing that the value of \( m \sqrt{f} \) fluctuates but only within a small range of 1.0-1.2. This result suggests that as a good approximation, \( m \) could be taken as 0.9 \( \sqrt{f} \) with the error being up to about \( \pm 10\% \) for the flow conditions concerned in this study.

Fig. 3 Friction factor evaluated based on Nikuradze’s data for turbulent pipe flows.
Physically Based Approach in Hydrology - What is the Benefit?

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INTRODUCTION AND HISTORICAL PERSPECTIVE

Advances in hydrology have traditionally relied heavily on empirical relationships that were derived from experimental data. With the advent of digital computers in the 1950s and 1960s, there was great optimism that modeling the entire hydrological cycle would allow significant advances to be made. The Stanford Watershed Model is such an example. However, the Stanford Watershed Model is a conceptual model that still relies on experimental data. As digital computers became more powerful in the 1970s and 1980s, governing differential equations such as the Saint Venant equations could readily be solved numerically. This advance sparked a proliferation of hydrological models that are called physically based models such as the Storm Water Management Model. This type of modeling was expected to facilitate monumental advances in hydrology. The outstanding feature in these models is that the values of the parameters are meant to be determined a priori (i.e. without any experimental data). However, over the past two decades, hydrologists have slowly realised that determining the values a priori with the current generation of physically based models is difficult if not impossible. Some of the difficulties are: (1) lack of areal measurement technique, (2) equifinality, (3) aggregation approach, and (4) over-parameterization. As such, the current generation of physically based models may be considered as conceptual models, which still require experimental data.

In view of the preceding, is there any difference between the physically based approach and the empirical approach, and is the physically based approach useful for hydrology? This article examines the benefit of the physically based approach through published results on the time of concentration of overland flow.

TIME OF CONCENTRATION OF OVERLAND FLOW

The overland time of concentration is an important parameter in many drainage design methods. Combined with the time of travel in a channel, it becomes the time of concentration of an entire basin. The latter is commonly used as a basis for the determination of the design discharge via rainfall intensity-duration-frequency curves. There are many overland time of concentration formulae available in the literature. For flow over a plane surface, these formulae are generally expressed in the following form:

\[ T_c = KN^aL^bS^cI^d \]  

(1)

where \( T_c \) is the time of concentration of overland flow, \( K \) is a constant, \( N \) is the retardance coefficient, \( a \) is the exponent to \( N \), \( L \) is the length of overland plane, \( b \) is the exponent to \( L \), \( S \) is the overland slope, \( c \) is the exponent to \( S \), \( I \) is the net rainfall intensity, and \( d \) is the exponent to \( I \). Table 1 contains the values of the exponents \( a, b, c, \) and \( d \) from eight empirical time-of-concentration formulae. It is apparent that for each exponent, different formulae generally give different values. So which value should be used? Can the differences be reconciled? Since the values are data-dependent, different data sets invariably produce different values. Further experimentation will not resolve the issue. If one of these formula is set for general use, the value of each exponent can only be arbitrarily fixed, and the empirical approach can never reconcile the differences. How can hydrology then move forward with a firm foundation?

However, with a physically based approach such as the kinematic wave method, deriving the values theoretically is possible even without any data. As an example, Woolhiser

<table>
<thead>
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<th>c</th>
<th>d</th>
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<td>Izzard (1946)</td>
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<td>0.333</td>
<td>-0.333</td>
<td>0.333 &amp; -0.667</td>
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<td>U.S. Army Corps of Engineers (1954)</td>
<td></td>
<td>0.55-(0.001/S)</td>
<td>-1.000</td>
<td>-0.430</td>
</tr>
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<td>- concrete surface</td>
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<td>0.467</td>
<td>-0.234</td>
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<td>Kerby (1959)</td>
<td>0.467</td>
<td>0.500</td>
<td>-0.333</td>
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<td>Federal Aviation Administration (1970)</td>
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<td>National Association of Australian State Road Authorities (1986)</td>
<td>1.000</td>
<td>0.333</td>
<td>-0.200</td>
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</table>
and Liggett (1967) used the kinematic-Manning approach to derive such a formula and the values for the exponents are given in Table 2. In this case, Woolhiser and Liggett’s formula can be set for general use, and the values for the exponents are fixed to those in Table 2. The experimental data can then be used to determine the value of the retardance coefficient for various surface types only. This physically based approach enables a firm foundation to be laid whereby hydrology can move forward with an agreed time-of-concentration formula.

CONCLUSIONS

The benefit of the physically based approach is that it can reconcile differences in empirical results and lay a firm foundation for hydrology to move forward.

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**Wave Breaking and Kinematics on a Reef-Top**

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S. Monismith

**INTRODUCTION**

Shallow, submerged reefs subject to significant wave action are common throughout the Pacific and Indian Oceans, e.g. the Great Barrier Reef in Australia. Such reefs often surround lagoons and low atolls and provide protection from incident waves by dissipating the wave energy through the surf zone on the seaward side of the reef. The location of the initial breaker zone on the seaward edge of the reef creates a potentially very wide surf zone in which wave setup and wave induced flow are significant in determining both water levels and mass transport on the reef-top. Hence, the hydrodynamics of reefs is being recognized as the principal formative agent for the sedimentary reef-top islands, circulation and flushing of the lagoon and also a critical factor in determining community distribution and production rates in coral reef ecosystems both by controlling the supply of nutrients and the level of turbulence on the reef [1-3].

In order to understand the wave breaking and the associated kinematics on a reef-top, an experimental investigation on an idealized two dimensional reef model is being carried out. The study emphasizes qualitative and quantitative description of the flow domain, which includes flow visualization, reef-top water level and velocity measurements.
EXPERIMENTAL METHOD AND MEASUREMENT TECHNIQUES

A schematic layout of the reef-model is shown in Fig. 1. The flow direction is from left-to-right and the ramp slope is 1:8. The experiments were performed in a wave flume 35 m long, 0.55 m wide and 0.6 m deep. In order to replicate an open system such as a platform reef, water in the wave flume recirculates through an external pipe that connects both ends of the flume. Regular waves are generated by a commercial wave-maker system from HR Wallingford. The signals from the calibrated wave probes are subsequently processed at a sampling interval of 0.02s and data acquisition duration of 100s. This input values determine the wave data recordings on the respective channel based on down-crossings of the running mean. The corresponding wave count is then used to obtain the necessary statistical parameter of mean water level \((h_{<})\) on the reef-top. Four wave probes were used to measure water level on the reef-top, which are spaced one meter apart as shown in Fig. 1; the calibration value of the probes has to be offset by an amount necessary to raise it to the reef-top from deep water.

Flow visualization was carried out by a digital SLR-camera; the object plane was illuminated by spot-light arrangement and the recorded images were subsequently processed. Velocity data was obtained by a two component Micro-ADV system, located at a distance of 1.6 m from the reef-edge. Data acquisition parameters of the ADV were properly tuned based on the expected maximum velocity range of the flow-domain. The present investigation considers a range of water depths \((h)\) from 35 cm to 45 cm and frequencies from 0.6 Hz to 1.4 Hz. The initial wave height \((H_o)\) varies from 4 cm to 18 cm.

RESULTS AND DISCUSSION

The images show that as the water depth increases from dry reef-top condition of 35 cm to 45 cm, wave breakup shifts from reef-face to reef-top. The higher depth case with the larger column of water on the reef-top can sustain a strong recirculation zone (Fig. 2b). In other words, before breakup, curling of waves dominates, which in turn gives rise to larger backward momentum to the water mass on the reef-top, and hence the fluid velocity may be negative. On the other hand, for lower water depth case, wave buildup takes place on the reef-face and hence breakup occurs in the upstream side (Fig. 2a). Consequently, the momentum associated with water particles on the reef-top will be significantly forward oriented with positive velocity. In order to substantiate this observation, the associated velocity field on the reef-top was characterized in Fig. 3.

Figure 2. Wave breaking over a reef-top for different water depths \((H_o=12 \text{ cm} & f=0.6 \text{ Hz}): (a) h=35 \text{ cm}; (b) h=45 \text{ cm}\)

(b)  

Figure 3. Stream-wise velocity on reef-top: (a) \(h=35 \text{ cm}\); (b) \(h=45 \text{ cm}\)

Note that the water depth case of \(h=35 \text{ cm}\) corresponds to initial dry condition on the reef-top. The measurement volume was restricted to the location of 1 cm from the reef bottom.
For the other depth, the measurement volume located at 2.5 cm from the bottom. The results indicate that as the water depth increases the velocity ($V_x$) decreases and becomes negative (Fig. 3b). Careful examination of the results shows that the velocity profile on the reef-top tends to become asymptotic with increasing wave height. This may be due to the saturation of the wave generator at high input values of wave height. The generated waves will not be clean, with initial breaking of the wave crest and presence of cross waves can significantly reduce the energy level of the waves before it reaches the reef-face.

Figure 4 shows the reef-top water level profile as a function of stream-wise locations ($L$) for the specific frequency of 0.6 Hz and for the respective water depths. The reef-top water level ($h_{reef}$) can be considered as an integrated measure of wave breaking and the associated kinematics on the reef-top. The results show that, for higher water depth case, reef-top water level does not change from its initial value of $10$ cm (Fig. 4b), while for lower depth case, appreciable increase of water level on reef-top from its initial dry condition (Fig. 4a). This behavior can be attributed to the dominance of backward (seaward) momentum for larger depth and forward (shoreward) momentum for smaller depth cases as explained previously based on qualitative visualization (see Fig. 2).

CONCLUSIONS

Based on the present experimental investigation, it can be concluded that the initial wet condition on the reef-top creates backward momentum that can mitigate water level increase due to wave buildup and breaking. On the other hand, dry initial conditions on the reef-top can enhance the water level because of the associated forward momentum.

REFERENCES


INTRODUCTION
Forecasting flows is important for water resources management. Previous studies [1] show that artificial neural networks generally demonstrate high predictive accuracy in forecasting river flows in medium and large watersheds. This study investigated the applicability of artificial neural networks in forecasting flows in small watersheds, i.e. the Bricklands sub-catchment. In addition to observed rainfall and flow data, the neural network developed include rainfall forecasts as input variables.

ARTIFICIAL NEURAL NETWORK
In this study, a multi-layer feed forward network with back propagation algorithm was chosen for forecasting flows at the outlet of the sub-catchment. The feed forward network was chosen as it has been demonstrated to be generally successful for the identification of complex non-linear rainfall-runoff relationships. The network consists of three layers, namely an input layer, a hidden layer and an output layer. The number of neurons in the input layer depends on the number of input variables used. The output layer contains only one tan-sigmoid neuron, producing the corresponding flow rates for a given input vector. The hidden layer contains a number of tan-sigmoid neurons.

SELECTION OF INPUT VARIABLES
A cross correlation technique suggested by [2] was used to identify the appropriate time lags of rainfalls to be included into the input vectors to the network.

Type I model was found to slightly overestimate the peak flows of high flow conditions (greater than 5.0m$^3$/s) as shown in Figure 3. This phenomenon is presumably due to the small amount of data points for high flows compared to the large number of data points in the training set for low and intermediate flows.
For forecast time of 1, 3 and 6 lags ahead, Type II models are generally capable of forecasting flows for the entire flow range. However, Type II models were found to be incapable of forecasting flows accurately once the forecast time equals or exceeds the time of concentration, as flow predictions were observed to be poor even for low and intermediate flow conditions.

This is because none of these rainfall events is included as input variables in the Type II models for forecast time with 8 and 10 time lags ahead of the current time. By physical reasoning, this is consistent with the notion the conventional ANN forecasting will be possible up to the time of concentration, but as demonstrated, the inclusion of rainfall forecasts can generally enhance its predictive capability.

**IMPACT OF ERRORS IN FORECAST RAINFAILS**

Through sensitivity analyses, the importance of rainfall forecasts in improving the predictive capability of the networks was investigated. These analyses further elucidate the maximum allowable error in the rainfall forecast, where once it is exceeded, inclusion of forecast rainfalls into the input vector does not improve the predictive accuracy of the networks.

For forecast time 1 lag ahead, the impact of errors in rainfall forecast on the network predictive capability was observed to be generally insignificant. This is expected as only one value of rainfall forecast is used. For forecast time 3 and 6 lags ahead, rainfall forecast seems to be capable of improving the network accuracy when its error falls within the range from -60% to +60%.

However, a much smaller range of error in forecast rainfall is required to preserve the applicability of the Type I models once the forecast time equals or exceeds the time of concentration. To preserve the applicability of the network for forecasting flows at 8 and 10 lags ahead, the error in rainfall forecast has to fall within the range from -35% to +25% and from -25% to +50% respectively. This dramatic reduction in error range was mainly due to the fact that rainfall forecast becomes more important in representing rainfall-runoff process as the forecast time increases.

**CONCLUSIONS**

This study demonstrates the applicability of artificial neural networks in forecasting flows in small watersheds with a concrete-lined drainage system.

The results of the study are summarized as follows:

1. The Type I models that incorporate rainfall forecast input are capable of forecasting flows for all the forecast times tested;
2. The Type II models are generally capable of forecasting flows for the entire flow range up to the time of concentration;
3. The allowable errors in rainfall forecast to preserve the applicability of the Type I models were found to decrease as the forecast time increases.

**REFERENCES**


Application of Single-Parameter Digital Filters for Hydrograph Separation and Development of Empirical Relationships

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Lim Wee Ho (WHLim@ntu.edu.sg)

INTRODUCTION
A reliable hydrograph separation method is necessary for surface runoff modelling and hydrological studies. This paper elucidates the separation characteristics of two single-parameter digital filters, which are referred to here as the “one-parameter” algorithm and the “conceptual method”. The filter algorithm that produces more reliable separation results is then adopted to separate the baseflow from the total runoff hydrograph for the Bricklands sub-catchment in Kranji. From the results of hydrograph separation, empirical relationships are developed for the study site. These empirical relationships relate total runoff depth, baseflow depth or direct runoff depth to total rainfall depth on both event and monthly basis. These relationships are useful for predicting baseflow depth, direct runoff depth and total runoff depth from the measured rainfall depth.

DIGITAL FILTER ALGORITHMS
“One-parameter” algorithm
The “one-parameter” algorithm [1] is expressed as:

\[ Q_{b,i} = \frac{k}{2-k} Q_{b,i-1} + \frac{1-k}{2-k} Q_i \]

for \( iib Q \leq Q_i \)

where \( Q_b \) and \( Q \) are the baseflow and total runoff; \( i \) is the time interval; and \( k \) is the recession constant during periods when there is no direct runoff.

“Conceptual method”
The “conceptual method” [2] is expressed as:

\[ Q_{b,i} = \frac{1-k}{2} (Q_i + Q_{i-1}) + kQ_{b,i-1} \]

for \( iib Q \leq Q_i \)

Using the traditional \( \ln(Q_t) \) versus \( t \) approach, recession constant was estimated for study site and found to be 0.988.

COMPARISON BETWEEN SEPARATION RESULTS
The two digital filters were applied for separating the 94 recorded hydrographs. For illustration purpose, separation results for a single-peaked and a multiple-peaked hydrographs are shown in Figures 1(a) and 1(b) respectively.

Figure 1. Separation results for a (a) single-peaked; (b) multiple-peaked hydrographs
The difference in baseflow separation between the two filters is generally not significant for the study site, as the baseflow hydrographs estimated from these two methods are close to each other.

To further elucidate the separation characteristics of these two methods, sensitivity of baseflow separation to recession constant $k$ was carried out, and the results are shown in Figure 2.

No significant difference in baseflow separation results could be observed between the two methods when $k$ value is larger than 0.98, where BF ratios between the two methods are close to each other. This explains the insignificant difference in baseflow separation results between the two methods for the study site (Figure 1), as $k$ value of the study site is about 0.988. For $k$ value smaller than 0.90, the BF ratio of the “one-parameter” algorithm tends to attain to its maximum value of 50%. In other words, the “one-parameter” algorithm is only capable of separating hydrograph up to 50% of the total measured hydrograph, implying that the algorithm is not applicable for catchments fed largely (more than 50% of the total runoff) by groundwater.

Figures 3 and 4 show the baseflow hydrographs for different $k$ values for the “one-parameter” algorithm and the “conceptual method” respectively. The “one-parameter” algorithm tends to produce an unrealistic sharp peak of baseflow right under the measured hydrograph peak when $k$ value is smaller than 0.96. No significant time lag between the measured hydrograph peak and baseflow peak was observed as indicated in Figure 3, implying that both the baseflow and surface runoff reach the stream simultaneously. As a matter of fact, this time lag should occur, as subsurface flow is generally much slower than surface runoff for the in-situ silty soils of the study site. On the other hand, a delay between baseflow peak and measured hydrograph peak was observed for the “conceptual method” (Figure 4) for all $k$ values.

**DEVELOPMENT OF EMPIRICAL RELATIONSHIPS**

Rainfall depth-total runoff depth, rainfall depth-direct runoff depth and rainfall depth-baseflow depth were developed for the study site for both event-based and on a monthly basis.

For event-based, the relationships are:

\[ D_T = 0.5386I_{\text{depth}} - 6.2644 \]  \hspace{1cm} (5)
\[ D_{DR} = 0.2571I_{\text{depth}} - 1.2081 \]  \hspace{1cm} (6)
\[ D_{BF} = 0.2815I_{\text{depth}} - 5.0563 \]  \hspace{1cm} (7)

where $D_T$ is the total runoff depth (mm); $D_{DR}$ is the direct runoff depth (mm); $D_{BF}$ is the baseflow depth; and $I_{\text{depth}}$ is the total rainfall depth.

For monthly basis, the relationships are:

\[ D_T = 0.6014I_{\text{depth}} - 21.276 \]  \hspace{1cm} (8)
\[ D_{DR} = 0.2502I_{\text{depth}} - 11.0333 \]  \hspace{1cm} (9)
\[ D_{BF} = 0.2770I_{\text{depth}} - 11.952 \]  \hspace{1cm} (10)
As the empirical relationships were developed based on a large number of hydrograph events (94 hydrograph events over a period of 18 months) and found to generally have $R^2$ values higher than 0.90, these relationships could hence be used reliably to estimate total runoff depth, direct runoff depth and baseflow depth from the measured total rainfall depth for the study site.

CONCLUSIONS

The “one-parameter” algorithm was found to have a maximum BFI value of 0.50, and hence is not applicable for catchments fed largely (more than 50% of the total runoff) by groundwater. For $k$ value smaller than 0.96, the “one-parameter” algorithm tends to produce an unrealistic sharp peak of baseflow right under the measured hydrograph peak. On the other hand, the “conceptual method” was observed to be able to provide realistic baseflow hydrographs for large baseflow separation. However, the “conceptual method” requires a reliable estimation of recession constant, as the method was found to be sensitive to recession constant for large baseflow separations. In addition, useful empirical relationships were developed to estimate total runoff depth, direct runoff depth and baseflow depth from the measured total rainfall depth for the study site.

REFERENCES


**ABSTRACT OF RESEARCH REPORTS**

**Ground Movement Induced Pile-Soil Interface Stresses**  
*Principal Investigator: Teh Cee Ing*  
*Report No: CEE/2006/160*

In conventional design, piles are designed to transfer superstructure load to the supporting soil strata. Increasingly, however, engineers begin to realize the importance of designing piles to resist loads induced by ground movements. The soil movements can arise from construction activities such as basement excavation, tunnelling, settlement due to ground water lowering or consolidation of underlying soft soil layer, and movements due to landslides. To arrive at a rational design approach, it is necessary to have an in-depth understanding of tile interaction between the pile and the moving soil mass. Apart from the relative pile-soil movements, the variation of interface stresses with soil movement is one key aspect which is currently not well-understood. The aim of this project is to determine the relationship between the soil displacement and pile-soil interface stresses through small scale model testings. Simultaneously, analytical and numerical modelling of pile problem will be carried out.

**Permeability & Pore Structure of Blended Cement Concrete**  
*Principal Investigator: Jong Herman Cahyadi*  
*Report No: CEE/2006/161*

Permeability of concrete is one of the most critical parameters in the determination of concrete durability. As the permeability of concrete is lowered, its resistance to chemical attack increases. In a broader sense, permeability encompasses the transportation of liquid as well as gas and vapour phases through the concrete. It has been reported that the incorporation of supplementary cementing materials such as fly ash, slag, silica fume, and natural pozzolans in concrete results in fine pore structure and changes to the aggregate/paste interface, leading to a decrease in permeability. However, there is no guideline for proportioning concrete to compensate for the effects of fly ash, slag, or silica fume. Moreover, the physical properties and chemical compositions of these pozzolans vary widely from place to place. In this project, blended cement concrete will be made with various water-cement ratio, pozzolans and proportioning. The permeability rate and the pore structures will be measured. A numerical model will be established to predict the pore structure of blended cement concrete. The relationship between pore structure and permeability rate will be proposed to take into consideration the connectivity factor.

**Novel Aerobic Granulation Biotechnology for High-Performance Industrial Wastewater Treatment**  
*Principal Investigator: Liu Yu (EN&WR)*  
*Report No: CEE/2006/162*

This research project is multidisciplinary in scope and will combine wastewater engineering know-how with microbiological and life-science techniques to investigate this aerobic granulation biotechnology. Experiments will focus on reactor operations, granule biodiversity and structure, and the development of this biotechnology for model industrial wastewaters.

**Analysis Of Microbial Water Quality**  
*Principal Investigator: Karina Gin Yew-Hoong*  
*Report No: CEE/2006/163*

The main objective of this study is to develop protocols for the rapid detection and enumeration of specific pathogens or their indicators, i.e. within 1-2 hours, using a combination of fluorescent probes and flow cytometry. Specific objectives are to: (1) Develop specific fluorescent probes for selected indicator organisms; (2) Develop flow cytometric protocols for analysis of these indicator organisms from a variety of water environments, including freshwater sources, tap water, ground water and reclaimed water; and (3) Compare concentrations of selected indicator organisms using flow cytometric techniques and conventional methods.

**Trip And Parking Generation Of Major Land Use Developments.**  
*Principal Investigator: Tan Yan Weng*  
*Report No: CEE/2006/164*

The objectives of this research are: (1) to select suitable land use development sites as study locations; (2) to obtain vehicle count and occupancy data along each driveway serving the selected sites and along roads adjacent to the sites; to collect parking data at carparks serving the selected sites; to obtain information on characteristics of each site; (3) to estimate trip and parking generation for each site (am peak, pm peak and 24 hours) and to relate this to site-related characteristics; and (4) to estimate the impact of MRT services on trip and parking generation. A total of 60 sites representing industrial, residential and other land uses will be selected for study. The research will involve the collection of characteristics of these sites, such as physical building size, number of employees, number of parking spaces, etc. Traffic surveys will be conducted on each site to obtained trip and parking data. The traffic surveys are essential to this project, and the work will be contracted to an external organisation.
Application of Real Options in Infrastructure Projects
Principal Investigator: Charles Cheah Yuen Jen
Report No: CEE/2006/165

The scope of the project includes a few case studies of international infrastructure development arrangements. The studies will help to (a) identify and characterize the managerial options found in selected infrastructure projects; (b) evaluate the most common real options identified using different modeling approaches; (c) assess the underlying assumptions, robustness and appropriateness of different modeling approaches as related to the matching characteristics of infrastructure projects studied; (d) develop guidelines for real option valuation and BOT contracting based upon the above results.

Prevention of Leaching of Cement Lining in Newly-laid Water Mains
Principal Investigator: Chui Peng Cheong
Report No: CEE/2006/166

This study is a collaborative research project between Nanyang Technological University and Public Utilities Board. The objective of the project is to identify and examine the factors leading to the leaching of cement mortar lining in newly-laid water mains, and to recommend measures to minimize or prevent its occurrence. The effects of cement matrix, pre-conditioning, physical barrier and characteristics of conveyed water were explored. An accelerated test using thin plate specimens and a pilot test on 300-mm pipe sections were conducted to investigate the leaching behaviour of cement mortar. In the present investigation, tap water, NEWater, distilled water and de-ionized water were used as the immersion test solutions for leaching. The measurements of pH, total alkalinity and calcium concentration were carried out to determine the water quality; whereas analyses based on the measurements of mass change and phosphate concentration, thermogravimetric analysis, mercury intrusion porosimetry and scanning electron microscopy were performed to examine the cement matrix, before and after the leaching tests. The results obtained for cement mortar with different compositions under various test conditions were assessed and compared, so as to provide guidelines on the use of cement mortar lining in newly-laid cement-lined water pipes.

Development of a Numerical Circulation Model for the South China Sea
Principal Investigator: Lo Yat-Man, Edmond
Report No: CEE/2006/167

The project is to develop a computational ocean circulation system suitable for modelling the current and flow patterns over a large oceanic area with the prototypical area being the South China Sea. The modelling system will be able to simulate the tidal current and ocean current caused by temperature/salinity gradients and wind forcing as well as current driving input from the surrounding areas. The project is funded under the MPA-NTU Maritime Research Centre and has a funding of $260,000 over duration of 3 years. The South China Sea including its approaching areas is one of the main navigation routes for Container Vessels and Very Large Crude Carriers. MPA is developing an enhanced Digital Tidal Atlas (DTA) with the aim of assisting shipping lines to improve their navigation safety in the Straits of Malacca and Singapore. Besides this, MPA has a commercial suite of coastal hydrodynamic codes under for predicting the local tidal and current patterns. These codes are, however, depth-averaged models. As such NTU is developing with MPA, a higher state-of-the-art numerical ocean model to handle deep sea/ocean flow accounting for coriolis and vertical variations. With this capability, MPA will then be able to extend the coverage of its DTA so as to assist shipping lines in ensuring navigation safety in the South China Sea. Lastly this will also help MPA extend its Search and Rescue Model and Oil Spill Model to be applied to the South China Sea. The system development will be executed in two fairly distinct phases; that of computational code development and user interface development. A public domain ocean circulation code, the Princeton Ocean Model (POM) will be extensively modified and used. A user interface will be developed so that the system can be effectively used by engineering community such as at MPA.

Clean Water Programme (CWP) Research
Principal Investigator: Tay Joo Hwa
Report No: CEE/2006/168

The Clean Water Programme (CWP) is a three-year joint effort between Nanyang Technological University (NTU) and Stanford University, and in collaboration with Public Utilities Board (PUB), to conduct research that will address Singapore’s pressing water needs. CWP is an integrated research plan that will investigate the feasibility of using Aquifer Storage and Recovery (ASR), Membrane Separation and Treatment (MST) and Photocatalytic Oxidation (PCO) for local water reclamation. It will develop advanced Water Quality Analytics (WQA) capabilities to boost Singapore’s water management and R&D infrastructure. The Agency for Science, Technology and Research (A*STAR) recently awarded NTU a $55.836 million CWP grant. These funds will be supplemented with NTU funds of $33.8 million to support a comprehensive research effort. A key goal of CWP is the continuing development manpower resources and research infrastructure of NTU. CWP has been designed to facilitate the transfer of expertise from Stanford to NTU and Singapore through frequent interactions between staff and students. NTU researchers and PUB staff will visit Stanford University to view and participate in ongoing Stanford research relevant to the CWP. Stanford University researchers will visit NTU and PUB to help develop local analytical capabilities, to assist in the design and implementation of field, laboratory and modelling studies, and to conduct regular workshops and seminars. CWP will spur Singapore’s efforts to be more self-reliant in water through the development of innovative technologies to produce good-quality water at low cost. It will also build up the capabilities of the local research infrastructure to provide state of the art engineering solutions and support large-scale, multifaceted, multidisciplinary projects. The participation of Stanford University testifies to Singapore’s growing stature as an environmental technology
hub and will position Singapore as a major player with the requisite technology and skills to address important environmental issues in the region.

**Rapid Detection and Enumeration of Bacteria and Viruses in Water Supplies**

*Principal Investigator: Gin Yew-Hoong, Karina*
*Report No: CEE/2006/169*

The main objective of this study is to develop protocols for the rapid detection and enumeration of specific pathogens or their indicators, i.e. within 1-2 hours, using a combination of fluorescent probes and flow cytometry. Specific objectives are to:
1. Develop specific fluorescent probes for the following indicator organisms (in order of priority): E. coli, total coliforms, bacteriophage and Enterococci.
2. Develop flow cytometric protocols for analysis of these indicator organisms (and others e.g. Cryptosporidia and Giardia) from a variety of water environments, including freshwater sources, tap water, ground water and reclaimed water.
3. Compare concentrations of selected indicator organisms and viruses using flow cytometric techniques and conventional methods.

**Active Operator Guidance System (AOGS) Module 4 – Development of Real-time Water Depth and Current Assessment System**

*Principal Investigator: Tan Soon Keat*
*Report No: CEE/2006/170*

The project aims to extract from the electronic navigation chart, safe depth (minimum water depth) that is passable to the vessel, given the time of passage, tidal water level, draft and bathymetry within a radius of a pre-selected x-ship length.

**Ship Interaction between a Tug Boat and a Container Vessel**

*Principal Investigator: Tan Soon Keat*
*Report No: CEE/2006/171*

The project aims to model and determine the attractive and repulsive forces generated by 2 vessels moving in close proximity to one another, whether in passing or overtaking manoeuvre.

**Research Study on Public Transport Accessibility and Its Impact on Travel (CQ2058)**

*Principal Investigator: Piotr Olszewski*
*Report No: CEE/2006/172*

This study aims to measure and examine public transport accessibility in Singapore. A key objective is to establish a relationship between public transport accessibility and trip generation as well as modal split. The study will involve field surveys to measure the physical walking accessibility to public transport in different residential zones in Singapore. The analysis of field data together with travel information obtained from LTA household interviews will allow to assess the catchment areas of MRT/LRT stations; to identify factors related to the pedestrian environment which deter people from using public transport and to recommend ways to improve public transport accessibility in the local context.
Fatigue Behaviour of Uniplanar Tubular K-Joints under Axial and In-plane Bending Loads
Candidate: Shao Yong-Bo
Report No: CEE/PhD/2006/105

In this study, a new modelling method is proposed for a uniplanar tubular K-joint containing an arbitrary surface crack located along the chord weld toe. This model has been proved to be efficient and effective in producing different quality mesh at different zones. The proposed model is used to analyze the stress intensity factors (SIFs) of any tubular K-joint. To validate the accuracy of these numerically computed SIFs, two full-scale tubular K-joints specimens were tested to failure under combined axial (AX) and in-plane bending (IPB) loads. The standard alternating current potential drop (ACPD) technique was used to monitor the rate of crack propagation of the surface crack located at the hot spot stress region. The experimental SIF results are found to be in complete agreement with the computed SIFs obtained from the generated models. Thereafter, parametric studies are carried out to produce 5,120 finite element models of tubular K-joints subjected under AX or IPB load. Based on these SIF results, a set of SIP equations for K-joints subjected to AX or IPB are proposed using regression analysis. The accuracy of the SIP values estimated from these parametric equations has been validated by comparing them with the numerical results. For design purposes, a new interpolation method is introduced, and it is implemented by developing a corresponding computer program. The error estimation of this method is also investigated, and it has been found that this alternative method to estimate SIF values is both flexible and efficient, and it can be extended for any other types of joints.

Experimental and Analytical Study on Lightly Reinforced Concrete Frames and Beam-Column Joints
Candidate: Tan Haiyang
Report No: CEE/PhD/2006/106

This research investigates the behavior of lightly reinforced concrete beam-column joints and wide beam-column joints under simulated earthquake excitation-reversed cyclic loading. The scope of this research includes experimental and analytical investigations and a parametric study on seismic behaviour of beam-column joints. Previous studies on beam-column joints are firstly reviewed. Two classical joint shear mechanisms are highlighted and also different failure modes are identified. In the literature review, three modern seismic design standards for design of beam-column joints are introduced with concerns of seismic provisions as well as the recommendations. An analytical study on the gravity-load designed reinforced concrete frames is carried out with comparing of the test results by other researchers. The experimental investigations include tests on individual lightly reinforced concrete interior beam-column joints with a slab and wide beam-column joints under reversed cyclic loading. The seismic behaviour of the tested specimens is discussed using the experimental results combined with the analytical results, which are obtained through a finite element analysis. The influence of variables on the seismic behaviour of beam-column joints is further discussed through a parametric study with aim to best understand the performance of beam-column joints under an earthquake attack. Based on the work presented in thesis, recommendations on design of lightly reinforced beam-column joints and future research are suggested.

Hybrid Connection in Precast Construction
Candidate: Sun Xin
Report No: CEE/PhD/2006/107

In this research, a new hybrid connection has been proposed to utilise the ease of steelwork installation for traditional precast concrete construction. The primary objective of this research is to study the behaviour and suitability of this kind of hybrid beam-column connection. An experimental programme was carried out to study the behaviour of hybrid connection under monotonic loading as well as cyclic loading. A total of twelve specimens comprising bare steel connections, partial hybrid connections and complete hybrid connections were tested. The experimental findings were used in developing the component-based mechanical model to predict the behaviour of hybrid connection under various loadings. A mechanical model and a component-based model have been proposed to predict the behaviour of single plate connection and hybrid connection under both monotonic and cyclic loading. The predicted moment-rotation relationships are incorporated into the connector elements in a finite element program for semi-rigid frame analysis. A simplified and yet reliable design method of hybrid connection as well as the design guide for the analysis of frames with hybrid connections have also been developed.

Relationship of Stiffness and Damping Ratio with Strain for Residual Soils
Candidate: Yeo Sir Hoon
Report No: CEE/PhD/2006/108

There is very little work done on characterization of soil stiffness and damping ratio with strain for residual soils in Singapore. This is because routine geotechnical design does not usually incorporate the non-linearity of soil stiffness and damping ratio is only needed for dynamic problems. However, stiffness-strain and damping ratio-strain relationships are important as engineering structures mobilises stiffness and damping ratio at different strain levels depending on the loading condition. To characterize the Singapore residual soils’ stiffness-strain and damping ratio-strain : relationships, pulse transmission tests (bender element tests and ultrasonic tests), cyclic simple shear and triaxial compression tests for shear strain ranges from 0.0005% to 5% were performed. In addition, the effects of soil parameters such as void ratio, confining
pressure, degree of saturation and loading conditions such as frequency and number of loading cycles on soil stiffness and damping ratio are investigated.

**Non-linear Dynamic Behaviour of Framed Structures subjected to Ground Motions**

*Candidate: Zhao Danfeng*

*Report No: CEE/PhD/2006/109*

The research work presented in this thesis proposes an approach as well as a design methodology for the dynamic nonlinear behavior of steel frames subjected to ground motions, induced by either earthquakes or underground explosions, with the aim of implementing a complete, accurate and effective program that is mainly used for moment resisting steel frames to consider both geometric and material nonlinearities. Besides, the behavior of steel frames subjected to ground shock, resulting from either underground explosion or accidentally abnormal loads has also been investigated. The major differences between the normal earthquake-induced loading and the shock-related vibration are highlighted. In addition, this presentation also deals with the response of structure due to ground motion as a process of energy transformation, which the proposed method has a great advantage on evaluating the various forms for energy with quantity.

**Integrated Genetic Search for Identification of Material Properties and Dynamic Excitations**

*Candidate: Wang Chao*

*Report No: CEE/PhD/2006/110*

This thesis develops a general back calculation algorithm based on genetic algorithm (GA), genetic programming (GP) and the proposed integration of GA and GP (inGAP) for identifying the dynamic properties of materials and the dynamic excitations of dynamic systems. With GA-based algorithm, the dynamic properties of hard rocks and aluminum foams have been correctly back calculated from the measured strain history of the strain gauge, which is cemented on the pressure bars of the proposed modified split Hopkinson pressure bar (SHPB) setups. In order to improve the search capability of the pure GP, GA is integrated with the pure GP as a local search method to develop the inGAP system. The proposed inGAP system is a general optimization and search solver. Based on any dynamic response of one degree of freedom, the proposed GP-based and inGAP-based identification methods can correctly identify the dynamic excitations acting on dynamic systems.

**Separation of Indium Cations from Semiconductor Wastewater Using Nanofiltration Membranes**

*Candidate: Wu Ming*

*Report No: CEE/PhD/2006/111*

Nanofiltration (NF) membranes were applied to the separation of both suspended InP particles and dissolved indium (In^{3+}) from the semiconductor wastewater. It was fund that under the optimized operating conditions, the NF treated water is ready for direct reuse. The results demonstrated the importance of electrochemical effects on NF performance. A semi-empirical model was established by correlating the In^{3+} rejection rate with the ability of In^{3+} to form complex under different operating conditions. The variation of pH in such a process was found of paramount importance affecting the interactions between the NF membrane and solution. The rejection mechanisms of NF of multivalent cations were also delineated. However, colloidal fouling of the NF membranes may have a negative impact on cost effectiveness of such a process. Two distinctive fouling layers were identified. The first layer is due to the deposition of InP particles while the second layer is mainly due to ionic interactions at the membrane surface.

**Analysis and Design of Blast-Resistant Structures**

*Candidate: Yang Guichang*

*Report No: CEE/PhD/2006/112*

The response of reinforced concrete beams and plates with an attached external layer of FRP plate/sheet on the tension face and subjected to air-blast loading are investigated using a simplified analytical model. The model comprises a number of distinct elastic phases, of which each phase terminates due to failure at a cross-section, and finally the plastic phase, when the reinforced concrete structure becomes a free movable mechanism. Campbell’s dynamic yield criteria is employed to take account of the strain rate effect of steel reinforcement. Closed-form formulas are derived to calculate the time when a cross-section yields using approximate numerical technique. With these formulas the time when a dynamic response phase terminates can be determined. Based on the simplified analytical model, the dynamic response of a reinforced concrete beam or plate is traced in accordance with a proposed algorithm. Full disclosure of the theoretical development is provided. A number of numerical examples are given to illustrate how the formulas and the algorithm can be used to analyze structures subjected to air-blast loading.

**Seismic Damage Assessment Based on Structural Response and Energy Approach**

*Candidate: Wang Yi*

*Report No: CEE/PhD/2006/113*

A study on seismic damage assessment using structural response and energy approach is presented in this thesis. Based on the quantitatively defined energy forms, a new structural damage assessment model including total damage potential index and local damage potential indices is proposed using modified force analogy method. The total damage index gives the overall damage conditions of a structure during an earthquake, while the local damage indices give insights into the locations of the structure where most damage has been suffered in the structure. The limit states of the proposed total damage index at different performance levels from minor damage to severe collapse are also drawn from probability theories. Applications of the proposed damage assessment model are then demonstrated in three aspects: 1. damage
Deterministic and Stochastic Dynamic Study on Inelastic Structures with Panel Zone Deformation
Candidate: Wang Zhe
Report No: CEE/PhD/2006/114

A simple analytical method of modeling the panel zone deformation is proposed in this research. The model of rigid end offsets is also included to give a more accurate structural model. The force analogy method combined with the state space method is used to analyze the inelastic dynamic response of structures under earthquake excitation. Numerical simulations are performed on a one-storey one-bay frame and a six-story moment resisting frame. In addition, Monte Carlo simulation method and reliability analysis are also performed on a six-storey moment resisting frame based on the proposed model. The significant influences of rigid end offsets and panel zone deformation on dynamic behaviors of inelastic structures are demonstrated in both deterministic and statistic sense. On the other hand, a new stochastic dynamic analysis method for inelastic structures based on the force analogy method is proposed for the first time. The advantage of the force analogy method in terms of time-saving is remained in stochastic procedure. The proposed stochastic force analogy method can produce the covariance functions of displacement, velocity, inelastic displacement, and plastic rotation at individual plastic hinge location. The reasonability of the proposed method is demonstrated by means of the agreement of results of the proposed method with those of the Monte Carlo simulation method.

Performance-Oriented Seismic Reliability Analysis of Structural Systems
Candidate: Gu Xiaoming
Report No: CEE/PhD/2006/115

There has been extensive research on the performance-based seismic design philosophy targeting a uniform seismic risk. However, a practical methodology of reliability-based seismic performance evaluation is far from mature. This thesis attempts to address some of the problems in performance-oriented seismic reliability analysis of structural systems. It contains four parts. In the first part, an extensive literature review is performed on related issues including performance-based design philosophy, seismic reliability analysis, approximate methods to estimate the seismic displacement demand, and the potential application of the fuzzy set theory. In the second part, probabilistic models defining three main performance levels are studied for both structural and non-structural components. The obtained structural and non-structural drift ratio limits show good correlations with relevant experimental data and code specified limit criteria. To incorporate both the structural and non-structural criteria into reliability analysis in a coherent manner, a fuzzy-random reliability model is developed as the third part of this study. Numerical application shows the proposed fuzzy-random model provides a comprehensive reliability evaluation incorporating the influence of both non-structural and structural damage on the building performance. Finally, a framework for establishing a probabilistic model of the maximum inter-storey drift in a framed structure is proposed for the purposes of design applications and rapid performance assessment. The proposed regularity index is found to have a good correlation with the drift amplification factor that defines the maximum inter-storey drift. An exponential equation is derived from regression analysis to describe the relationship between the amplification factor and the regularity index.

Adaptive Refinement Analysis for the Coupled Boundary Element Method – Reproducing Kernel Particle Method
Candidate: Shuai YingYong
Report No: CEE/PhD/2006/116

The boundary element method (BEM) and the meshless methods have been applied in many engineering problems. In this study, a review is investigated to provide background knowledge on the \textit{a posteriori} error estimation and adaptive refinement for the BEM, meshless methods and the coupled method of the BEM and meshless methods. A new coupled method of the BEM and the reproducing kernel particle method (RKPM) is proposed for the two dimensional elastostatic problems to improve the solution efficiency. Numerical experiments on four benchmark problems are also carried out to compare the convergence rates of the BEM, the RKPM and the coupled BE-RKPM. A new \textit{a posteriori} error estimator for the RKPM is constructed by combining the Zienkiewicz and Zhu (Z-Z) error estimation with a new stress recovery procedure. Numerical examples are reused to test the efficiency of the new \textit{a posteriori} error estimator. The automatic adaptive refinement procedures for the BEM, the RKPM and the Coupled BE-RKPM are suggested for the 2D elastostatic problems. Numerical studies are then carried out to test the effectivity of the suggested adaptive strategies by comparing with the results of the uniform refinement.

Structural Behaviour Of High-Strength Rectangular Concrete-Filled Steel Hollow Section Columns
Candidate: Liu Dalin
Report No: CEE/PhD/2006/117

This thesis presents an experimental and analytical investigation on the behaviour of high-strength rectangular concrete-filled steel hollow section (CFSHS) columns. In the experimental programme, a total of 117 columns are tested under concentric loading, pure bending and eccentric loading. The squash load of stub CFSHS columns provides an upper limit of the axial load capacity of CFSHS columns. The test data obtained from the pure bending tests, on the other hand provide the flexural strength and stiffness of CFSHS columns. The effects of load eccentricity and slenderness on the
ultimate capacity of CFSHS columns are investigated in the eccentric loading tests. In the analytical study, three models are proposed for the numerical analyses on CFSHS columns under axial loading, pure bending and eccentric loading. The calibration of the models against the test data suggests that they can accurately predict the ultimate capacity of high-strength rectangular CFSHS columns. Based on the experimental and analytical studies, guidelines are developed for the design of high-strength rectangular CFSHS columns.

Behaviour of Reinforced Concrete Short Columns Subjected To Multi-Directional Loading
Candidate: Nguyen Xuan Hoang
Report No: CEE/PhD/2006/118

In this study, nine identical rectangular columns were tested in shear, torsion or combination of them under high axial load. The columns under shear force behave very brittle with the drift ratio at failure around 1%. Under high axial load, biaxial shear capacity seems to follow the elliptical interaction curve applicable for lower axial load. The interaction between shear and torsion can be approximated by a circular curve. For short columns under torsion, arch action is significant and thus, spaced truss analogy is not very appropriate. Additionally, behaviour of RC short columns under complex loading was simulated by finite element method. The three-dimensional cracked concrete model developed in University of Tokyo was modified. With modifications the finite element analysis can simulate behaviour of short columns in terms of load carrying capacity, load-deflection envelope, failure mode, strain of reinforcement. The results were insensitive to mesh size of finite elements. Furthermore, the shear design method proposed by Architectural Institute of Japan (AIJ) was modified. The modified method provides better prediction of shear capacity of 38 columns tested previously. The method is more rational, reliable and simpler. The method is not only applicable for short columns but also likely applicable for slender columns.

Performance-Based Blast Resistant Design Of Reinforced Concrete Frame Structures Under Distant Explosions
Candidate: Rong Haicheng
Report No: CEE/PhD/2006/119

Numerical finite element analyses to evaluate the structural dynamic responses are performed on a six-storey reinforced concrete frame structure under distant explosion conditions. The finite element models involved are discussed and experimentally verified. Results show that the whole dynamic response process of the frame structure can be approximately divided into two stages. Localized responses of the blast-loaded members are critical at response stage I while the global responses of the structural system dominate at response stage II. To keep the localized responses of the blast-loaded members under control in meeting the expected performance level, a performance-based blast resistant design approach for reinforced concrete structural members is developed with the energy spectra. Given the objective performance level defined by the combination of target displacement and target displacement ductility factor, a design procedure to uniquely determine the effective depth and longitudinal reinforcement ratio of the member is addressed and implemented into the practical design examples. The accuracy of the approach in terms of controlling the responses of maximum displacement and displacement ductility factor for the designed member under the given blast loading is numerically evaluated by comparing them with their respective design targets. Two error indices quantifying the errors between the responses of designed member and the design targets are defined and their simplified formulae are derived with extensive numerical studies and statistical analyses. The applications of the error indices formulae into the modification of the developed design approach as well as the probabilistic performance assessment of the designed member under blast loadings are exhibited. To ensure the global responses in terms of maximum inter-storey drift ratio (MIDR) at the expected global performance level, a new design method is presented for a multi-storey reinforced concrete frame structural system based on the equivalent static force (ESF). The calculation model of the ESF that produces the same MIDR effect as that under the distant blast condition is discussed. Several design examples are employed to demonstrate the method implementation while verifications of the MIDR responses of the designed frame structures are carried out with numerical dynamic analyses.

IFC (Industry Foundation Classes) - based information model for the integration of the Structural Design and Architectural Design
Candidate: Wan Caiyun
Report No: CEE/PhD/2006/120

In order to change the traditional ‘human-interpreted’ communication and improve the capability of interoperation and the efficiency of information communication, an IFC-compliant information model for building design processes and an integrated building design system is developed in this research. First, the process-oriented information modeling (PoIM) methodology is developed as the creative way to analyze information requirements and develop IFC extension models. Through the integration of IDEF0 process models and enhanced IDEF1 information models, a comprehensive view of the information is provided and communication becomes more effective and efficient. The proposed methodology is used to build the genetic information model for structural analysis and design domain. The usefulness of IFC to meet the needs of structural analysis is assessed from two levels and perspectives: 1) a generic and conceptual level from professionals’ knowledge; 2) the detailed requirements of structural analysis applications. The necessary IFC extensions for these information gaps are then developed. Finally, an IFC-based web-enabled integrated building design (IWIBD) system is developed, where a model server is utilized to support both IFC-based data integration and transaction-based cooperation between architectural design and structural analysis processes. A prototype system has been implemented and two case studies are carried out to demonstrate its validity and feasibility. It is hoped that with
Strain Softening and Instability of Sand under Plane-Strain Conditions

Candidate: Dariusz Wanatowski
Report No: CEE/PhD/2006/121

The majority of experimental studies on strain softening and instability behaviour of sand have been carried out under axisymmetric conditions. However, the plane-strain condition is the most common practical situation in geotechnical engineering. Therefore, it is more relevant to study the stress-strain behaviour of soil under plane-strain conditions. A comprehensive experimental study on the strain softening and instability behaviour of sand under plane-strain conditions is presented in this thesis. A new plane-strain apparatus was developed and used in this study. A granular fill, the so-called Changi sand, was retrieved from a reclamation site in Singapore and used in the experiments. The drained and undrained stress-strain and strength of Changi sand under plane strain were studied. The critical state line, the failure lines and the instability lines were determined. The tests were conducted under both deformation-controlled and load-controlled loading modes so that the effect of loading mode on the strain softening and instability behaviour could be studied. The test results show that the post-peak behaviour of sand is affected by the loading mode. Under a deformation-controlled loading mode, strain softening develops, whereas under a load-controlled loading mode, instability occurs. It has also been observed that the critical state line under the plane-strain condition is different from that under the triaxial condition. In stress path controlled tests, strain softening in the form of decreasing deviatoric stress may be suppressed. In this study, the strain softening behaviour of sand was investigated by strain path testing, in which \( \frac{d\varepsilon}{d\varepsilon_v} \) was controlled. Under plane-strain conditions, two types of strain softening, material softening and banding softening, were identified. The occurrence of strain softening under plane-strain conditions is affected by the void ratio, the imposed strain increment ratio, \( \left( \frac{d\varepsilon}{d\varepsilon_v} \right)_i \), and the resultant asymptotic stress ratio, \( (q/p')_{\text{as}} \), has been established experimentally. This relationship is consistent with that established by Chu et al. (1992) under axisymmetric conditions. Following the previous studies of Leong (2001) and Loke (2004) under axisymmetric conditions, the instability behaviour of both contractive and dilative sand was further studied under various drainage conditions along a constant shear stress path with decreasing mean effective stress. The results have shown that the instability line defined using Lade’s method, is the same for both drained and undrained conditions and the instability line defines the lower bound of all the possible unstable conditions regardless of the drainage conditions. Therefore, a zone of instability can be defined as the area bound by the failure line and the instability line. Instability will occur under either undrained or drained condition when the stress state falls into the zone of instability. A runaway type of instability can occur even for medium dense sand when a constant shear test is conducted under a constant \( \frac{d\varepsilon}{d\varepsilon_v} \) condition (CSSP test). Similarly to the condition for the occurrence of strain softening, the strain increment ratio imposed to a specimen has to be adequately low (i.e., negative) to generate pore water pressure. If the \( \left( \frac{d\varepsilon}{d\varepsilon_v} \right)_i \) experienced by the specimen in a strain path test is more negative than the \( \left( \frac{d\varepsilon}{d\varepsilon_v} \right)_\text{as} \) obtained from a drained test, the specimen will become unstable. The increase in pore water pressure is the necessary condition for the occurrence of this type of instability. It is found that the instability line obtained from the CSSP tests coincides with the peak stress line obtained from the strain path tests conducted under deformation-controlled loading mode. This shows that the peak stress line can be used to predict the instability conditions in CSSP tests. Shear bands will instantly occur during plane-strain tests on medium loose and medium dense sand under drained conditions. However, shear bands do not always occur when on very loose sand is tested under drained plane-strain conditions. Under undrained conditions, shear bands can also develop unless the specimen is looser than the void ratio at the critical state. The measured shear band inclinations for the Changi sand are in the range from 49.6° to 58.6°, giving an average value of 56°. Shear band orientations were best predicted by Roscoe’s solution.

Development of Microbial Granules under Alternating Aerobic-Anaerobic Conditions for Carbon and Nitrogen Removal

Candidate: Qin Lei
Report No: CEE/PhD/2006/122

This study comprised four parts. The first part investigated the effect of settling time on aerobic granulation in SBR, and results showed that a short settling time would favour aerobic granulation. The part of the study led to the development of a novel selection pressure theory for aerobic granulation. The second part looked into microbial granulation under alternating aerobic-anaerobic conditions for carbon and nitrogen removal without supply of external carbon source. It was found that microbial granules were successfully formed at NLR of 0.15-0.45 kg N m⁻³ day⁻¹, and 95% of influent COD and 24-50% of nitrogen were removed. To improve the nitrogen removal efficiency, the third part of this study, ethanol was supplied as external carbon, and this strategy resulted in a complete denitrification at all the NLRs, i.e. over 99% of influent nitrogen was removed. The last part of the study examined the feasibility of denitrification on PHB in microbial granular sludge SBR. Results showed the limiting capacity of PHB as reducing power for denitrification by microbial granules.

Stress and Strain Concentrations of Completely Overlapped Tubular Joints

Candidate: Gao Fei
Report No: CEE/PhD/2006/123

A uni-planar completely overlapped tubular joint specimen is tested under lap brace axial (AX), in-plane bending (IPB), out-
of-plane bending (OPB) and combined loadings to determine the SCF at the joint intersections of the chord and through brace and the through brace and lap brace. The experimental results revealed that the curve fittings of the strain values within the extrapolation region are fairly linear. The maximum SCF occurs at the saddle of through brace near the lap brace for the joint under AX and OPB, while at the crown heel of lap brace for the joint under IPB. For the joint under combined loadings, the maximum HSS can occur between the crown and the saddle of members. In the finite element (FE) analysis, both 8-node thick shell and 20-node solid elements are suitable for modelling the joint specimen. The SCF obtained from the FE models agree very well with those from the test specimen. In view of the computational time, the 8-node thick shell FE model is more suitable for the parametric study of completely overlapped tubular joints. A total of 5184 FE models of the joint are created with wide range of geometrical parameters. A set of parametric equations is proposed for predicting the SCF of completely overlapped joints under basic load cases. The equations are verified against the acceptance criteria of Fatigue Guidance Review Panel. The assessment of the proposed equations is based on the database of FE analysis and test results. The completely overlapped joint is commonly interpreted as two separate T/Y joints. However, the existing T/Y-joint parametric equations cannot accurately predict the SCF of completely overlapped joints under AX and OPB loadings. The T/Y-joint parametric equations only predict reasonably well the SCF of the joint under IPB loading. For this load condition, the gap size variation on the effect of SCF of completely overlapped joints is negligible.

Seepage Effects On Geometry And Turbulence Flow Characteristics
Candidate: Lu Yan
Report No: CEE/PhD/2006/124

This study explores how seepage affects the angle of repose of cohesionless sediments, geometrical dimension of dunes, and turbulence characteristics of flows over a stationary two-dimensional dune. Experiments were conducted to achieve the stated objectives except that a theoretical analysis was also carried out to examine seepage effects on the angle of repose of the sediment. In the study, a total of four series of tests were conducted, each with a definite objective. The angle of repose of sediments in the presence of seepage can be computed using a theoretical expression. The derived relationship is then reasonably applied to predict the lee-side slope of dunes when the applied seepage rate is known. The variations of the height and length of dunes caused by seepage were then analyzed; they show that the dune height is reduced when injection is present; while suction increases it. Additionally, the response of dune celerity to seepage is also examined. The data show that suction increases the celerity while injection reduces it. The response of the time-average velocity distributions, turbulence intensity distributions and Reynolds shear stress distributions over a single fixed, two-dimensional dune to bed suction was investigated. The results confirm that suction can render the turbulent flow in the boundary layer to be more “laminar” or less turbulent.

Investigation of Secondary Flows in Open Channel with Longitudinal Bedforms
Candidate: Wang Zhiqian
Report No: CEE/PhD/2006/125

Characteristics of open channel flows can be significantly modified in the presence of secondary flows. This study aims to investigate time-mean characteristics of open channel flows subject to steady large-scale longitudinal vorticity, which are generated by longitudinal bedforms. Laboratory experiments are conducted for six types of bed configurations. The results demonstrate that the generated secondary flows generally appear as pairs of counter-rotating circulations across the primary flow. The streamwise velocity that is significantly modified by the cellular secondary flows can be well represented by a function proposed in the log-wake form. The analysis shows that the modified primary flow can be linearized so that a flow quantity can be generally decomposed into two components, one being related to the average base flow and the other symbolizing the perturbation caused by the bed configuration. This study also discusses effects of cellular secondary flows on the concentration distribution of suspended sediment.

Shear Strength and Volume Change Relationship for an Unsaturated Soil
Candidate: Trinh Minh Thu
Report No: CEE/PhD/2006/126

Shear strength of unsaturated soil is commonly obtained from a consolidated drained (CD) triaxial test. However in many field situations, fill materials are compacted where the excess pore-air pressure developed during compaction will dissipate instantaneously, but the excess pore-water pressure will dissipate with time. It can be considered that the air phase is generally under a drained condition and the water phase is under an undrained condition during compaction. This condition can be simulated in a constant water content (CW) triaxial test. Comparisons between the shear strength parameters obtained from the CW and the CD triaxial tests have not been extensively investigated. An elasto-plastic model for unsaturated soil with the incorporation of soil-water characteristic curve (SWCC) was proposed in this study. The proposed model was verified with experimental data. A series of SWCC, isotropic consolidation, the CW and CD triaxial tests were conducted on statically compacted silt specimens in a triaxial cell apparatus. The experimental results from SWCC tests under different net confining stresses showed that the air-entry value and the yield suction increased nonlinearly with the increase in net confining stress. The results of the isotropic consolidation tests indicated that the yield stress increased with the increase in matric suction. The slope of the normally consolidated line (NC), the slope of the unloading curve and the intercept of the consolidation curves at the reference stress decreased with the increase in matric suction. The results indicated that the effective angles of internal friction, $\phi'$, and the effective cohesions, $c'$, of the compacted silt as obtained from both the CW and CD tests were identical. The results of
Damage Assessment of Reinforced Concrete Frames To Ground Excitations

Candidate: Lim Chee Leong
Report No: CEE/PhD/2006/127

Within a multitude of hazards that can lead to structural failure, explosion-induced ground motions (EIGMs) is one. When subjected to EIGMs, the structure experiences small structural deformations and large induced shear forces during the loading duration (Phase I). After the loading duration, larger structural displacements are experienced during the free vibration duration (Phase II). While the local mode response is significant during Phase I, this is less significant during Phase II. For reinforced concrete (RC) frames subjected to EIGMs, research is conducted related to the response analysis and damage assessment. During Phase I, the numerical examples have shown that Euler-Bernoulli elements can produce accurate response results as compared to Timoshenko elements. This is true when a sufficient number of elements is used, and the mass of the column is distributed at intermediate nodes. During Phase I, the shear forces and bending moment induced are the largest at the column base. This is regardless of the boundary condition at the top end of the column as demonstrated using a fixed-fixed and a fixed-free column. An approximate method is proposed for the response analysis during Phase I. The approximate method is developed for a system of multiple degrees of freedom so that local mode response can be included. Furthermore, it produces a time history for the response, which is important to capture the peak forces during Phase I that can occur during the loading duration. The proposed approximate method produces accurate response results during Phase I. Further, the approximate response obtained at the end of Phase I can serve as the initial conditions for the response analysis of Phase II. In short, the approximate method can lead to substantial savings in computational effort by separating response analyses of Phase I and Phase II. For the damage assessment, research is conducted for damage assessment during Phase I and Phase II. In this thesis, it has been shown that the shear strength of an RC section may increase by about 40% under EIGM loading. A damage assessment technique is proposed for response of RC frames during Phase I. This technique considers the strength-enhancement effects and reflects the possible failure mechanisms. This is applied for a structural-level model, and the results are compared with those from a material-level model. The proposed damage assessment method produces damage results similar to those from a material-level model. In addition, the proposed damage assessment method can reflect the failure mode resulting from direct shear, which was not incorporated in the material-level model used for comparison. For the damage assessment during Phase II, a failure mode at beam-column joints has been incorporated via a limiting inter-storey drift ratio based on experimental observations. This joint failure mode has not been considered by other damage assessment techniques. A simplified method for damage assessment of an RC frame has been proposed. The simplified method shows that the damage of an RC frame subjected to EIGMs can be determined by performing the damage assessment on the column at the first storey. Thus, the damage of an RC frame can be accurately assessed in a shorter time and requires a substantially less computational effort.
borehole traveling through single joint and multiple parallel joints. Along the special direction normal to the joints, prediction models for estimating the maximum rebound ratio are also proposed. The proposed prediction models are verified to be reliable by applying them to an explosion test in Mandai, Singapore.

**Model Updating and Structural Assessment Using Vibration Data with Artificial Intelligence Algorithms**

*Candidate: Tu Zhenguo*

*Report No: CEE/PhD/2006/129*

Catastrophic structural failures have been observed in civil infrastructures worldwide due to the dramatic changes of effective structural parameters. The prevention of such structural failures requires timely evaluation of the structural conditions throughout their life span. The FE model updating techniques have evolved for purpose to provide a realistic analytical model for a given structure so that its present condition can be assessed and the remaining service life may also be predicted under foreseeable loading conditions. In view of the limitations of the conventional model updating methods, this research programme aims at improving FE model updating techniques in such crucial aspects as less demand on the form of measurements, noise-resisting ability, and avoiding local solutions. In addition, appropriate forms of the measurement data and effective expansion of the response dataset are also investigated, in conjunction with appropriate algorithms, in order to ensure a more reliable updating outcome.

**Ultimate Strength and Fracture Behaviour of Cracked Welded Square Hollow Section T-Joints**

*Candidate: Yang Zhengmao*

*Report No: CEE/PhD/2006/130*

In accordance with the full-scale tests results of cracked square hollow section (SHS) T--joints, an actual numerical model of the cracked SHS joints is constructed. Based on this model, an automatic mesh generation program is proposed. This model is then used to refine the parametric study of the effect of the variables, such as the crack shape, crack size and the weld. Based on the yield line theory, the plastic collapse loads of cracked SHS T-joints under brace end axial loads are derived and validated experimentally and numerically. It is found that the yield line theory gives reliable prediction of the plastic collapse strength for the cracked SHS T-joints with medium $\beta$ ratio. Furthermore, in accordance with the $J$-integral approach the failure assessment diagrams (FADs) of the SHS T-joints are constructed subsequently. It is confirmed that the standard BS7910 (1999) Level 3A FAD is appropriated for determining limits of safe loading of cracked SHS T-joints under brace end axial loads provided the plastic collapse load is calculated using the conservative yield line formulae neglecting the influence of the welds and the chord wall thickness.
PUBLICATIONS

Publications of academic staff in journals and conference proceedings during the period from 1 July 2005 to 30 June 2006. Authors who are not members of the School are marked by *


PUBLICATIONS


Sachs, T. and Tiong, R., 2005. Political risks and investment decisions in PPPs – A proposed framework for determining PPP structures that satisfy an investor’s decision criteria with respect to the political risks. *Proceedings of the Queensland University of Technology, Research Week, 4-8 July 2005*, pp. 1392-1402.


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CIVIL ENGINEERING RESEARCH

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INNOVATIVE CIVIL ENGINEERING RESEARCH IN UNDERGROUND TECHNOLOGY AND ROCK ENGINEERING

Civil engineers have always been called upon to develop the infrastructural system of societies. Sometimes, this call in the modern world is extended beyond the traditional aspects of the profession because of other imposed conditions. In Singapore for example, the lack of land can be a serious barrier to the economic and social development of the city state. The need to create space to satisfy demands of the society has pushed the frontier of civil engineering research and practice to a level beyond the norm. To satisfy the thirst for space, innovative ideas have been sought to utilize underground rock caverns as an alternative to above-ground land space. In order to integrate such space for everyday living and utilization, much research is necessary. To this end, the School has embarked on an extensive research program that incorporates many aspects of civil engineering disciplines to ensure a seamless integration of underground space utilization, safety and health.

In order to have an overall understanding on how underground space can be successfully integrated as part of the infrastructural system of the country, the school is presently hosting an underground technology and rock engineering research program (UTRE), which is sponsored by Defense Science and Technology Agency. The objectives of the UTRE are:

1. To offer professional consultancies for underground space development in Singapore;
2. To conduct manpower training, technology transfer, and develop international collaborations.

The structure of the UTRE programme is shown in Fig. 1. Four major research areas are: (A) Rock Dynamic Testing and Constitutive Models; (B) Modelling, Design and Construction; (C) Subsurface Infra-Structure Planning and 3D-GIS; and (D) Surface and Urban Environmental Issues and Structural Monitoring.

A. Rock Dynamic Testing and Constitutive Models

The objective of this research component is to investigate the dynamic properties of different rocks and to develop dynamic constitutive models that are suitable for numerical simulation of underground structures subjected to blast and/or impact load. To this end, split Hopkinson Pressure bars and gas guns...