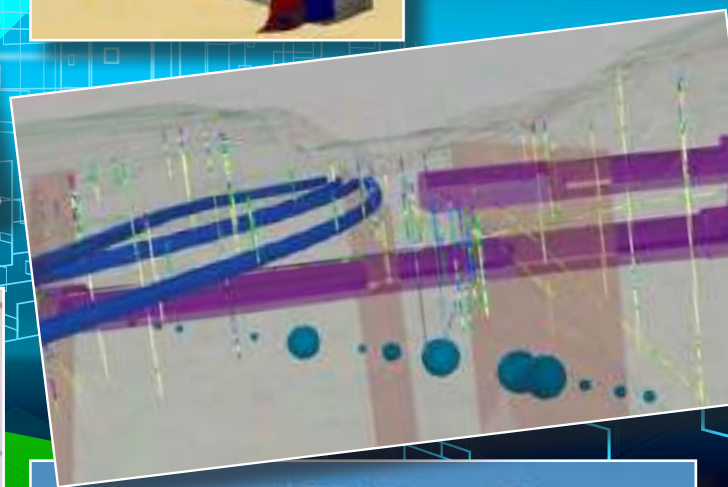
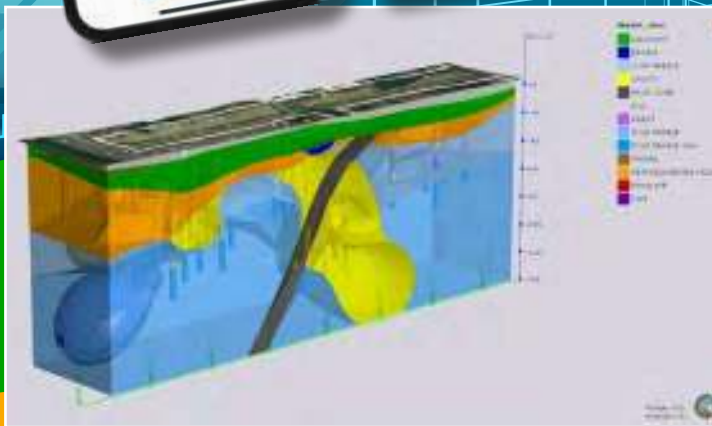
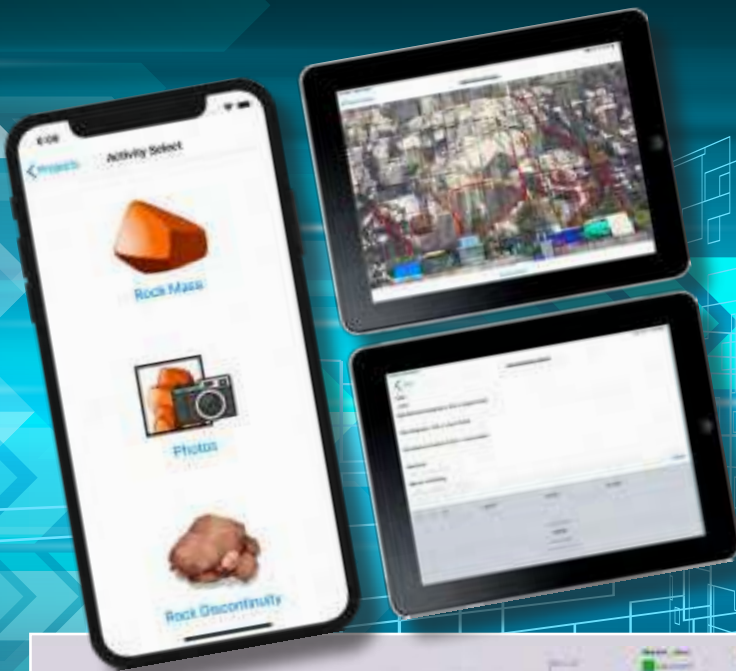


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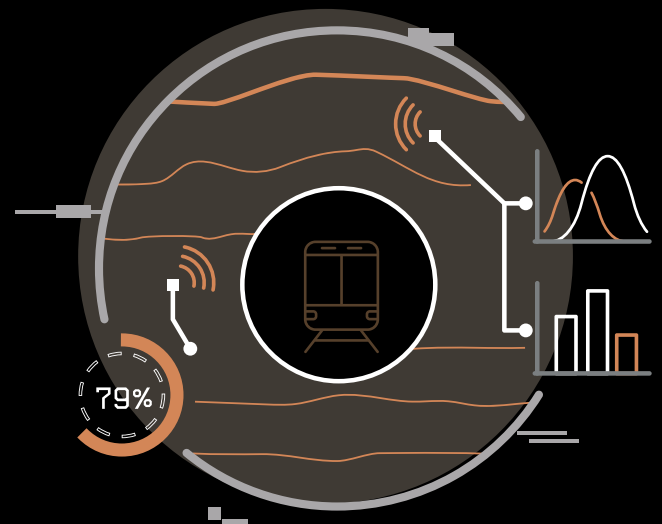
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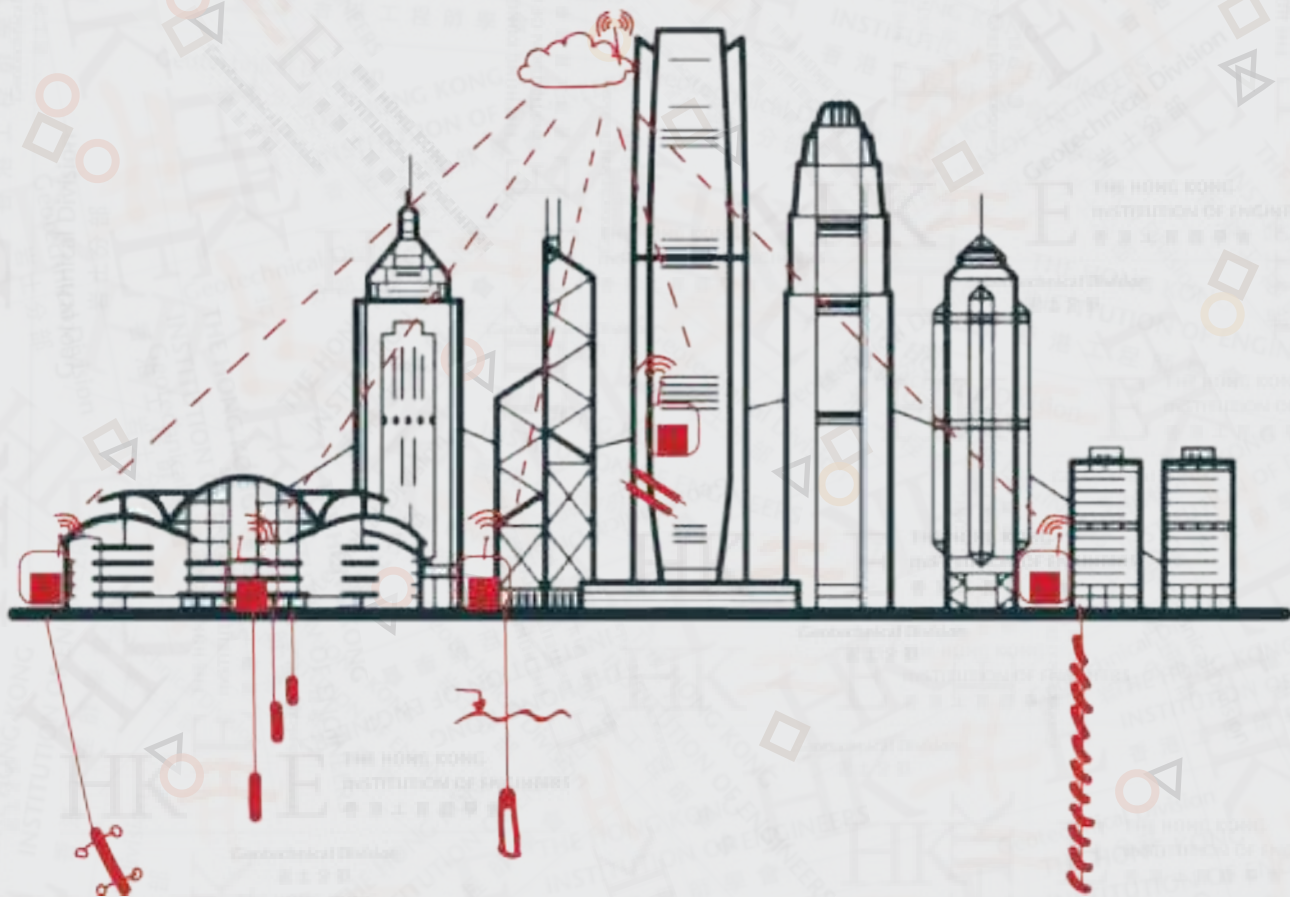


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Geotechnical Division, The Hong Kong Institution of Engineers

Transformation in Geotechnical Engineering – Technology, Digital and Innovation

11 April 2019
Hong Kong

Jointly organised by:
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Captions of Figures on the Front Cover

Clockwise from top left:

1. Examples of Mobile Phone/Tablet-based Discontinuity Survey Tools. Arup's 'RockSlope' App
2. Shotcrete thickness analysis carried out by ADAM Tech 3DM Analyst
3. Example of 3D Geological Model for a tunnel project
4. Examples of Survey Tools for the Generation of Point Cloud Datasets. Photogrammetric Data Collection Tools (©DJI)
5. Geological model generated by 3D implicit modelling software



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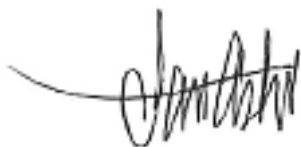
FOREWORD

I would like to welcome all distinguished guests, speakers, academics and geotechnical professionals to the 39th HKIE Geotechnical Division Annual Seminar.

The 2019 Seminar is titled “Transformation in Geotechnical Engineering – Technology, Digital and Innovation”. It is themed on advances in technology and innovation in geotechnical engineering with particular focus on digital transformation. Evolving digital platforms and sensor technologies together with advances in engineering theory, testing and analysis, design methodology, and construction techniques are providing geotechnical engineers with new insights and capabilities to deliver better project outcomes.

With the aim of generating useful insights for the profession, the HKIE Geotechnical Division Annual Seminar is a platform for practitioners to share innovations, ideas, and techniques in geotechnical engineering to pursue higher goals and meet new demands in Hong Kong and overseas. We are particularly grateful to Dr. Nick O’Riordan, the 58th Rankine Lecturer and Arup Fellow, who has agreed to deliver the keynote lecture on Some Aspects of Dynamic Soil Structure Interaction: Excerpts from The 58th Rankine Lecture. I am also delighted Mr. Leslie Lu, General Manager for Digital Transformation at Cathay Pacific Airways, is providing a perspective on how digital advances are being harnessed outside of the construction sector.

On behalf of the Geotechnical Division, I would like to express my gratitude to the invited speakers and authors contributing to this seminar. My special thanks go to all members of the Organising Committee, led by Ir. Jack Yiu, who have worked extremely hard in making this event happen. I would also like to thank the Hong Kong Geotechnical Society (HKGES) for jointly organizing this seminar, as well as the HKIE Civil Division and Structural Division for their support.



Ir Ian Askew
Chairman, Geotechnical Division
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TABLE OF CONTENTS

PAPERS	Page No.
1. Some aspects of dynamic soil-structure interaction: excerpts from the 58th Rankine Lecture <i>N.J. O’Riordan</i>	1 – 19
2. Application of Digital Engineering to Geotechnical Tunnel Design <i>T.H. Chan, P. Woodmansey, H.Y. Suen, S.W. Lee, T. Henderson, K.K Wu</i>	20 – 28
3. The Slope Greening by Tree Planting on a Soil Cement Fill Slope in Hong Kong <i>Morris C. K. Cheung, S.L. Chiu, Chester S. C. Leung</i>	29 – 35
4. A Review of Advances in Shallow Seismic Investigation for Managing Ground Risk <i>R. Eddies, E. Cox, M. Fliedner, T. Bailly, T. Chauveau, B. Barnwell, Y. Benboudiaf</i>	36 – 46
5. Quantification of the Risk in the Mable Sites by Geostatistical Analysis <i>S. Gao</i>	47 – 54
6. On The Application Of UAVs For Rock Joint Mapping – A Case Study <i>C. Gibbons, O. Hoel, K.P. Lo, S.L. Chiu, K.W. Yan, C.F. Lau</i>	55 – 61
7. Impact of Soil-Structure Interaction on the Seismic Design of Large LNG Tanks <i>A.S. Hokmabadi, E. Leung, M. So, J. Yiu</i>	62 – 70
8. A ‘whole-of-life’ approach to project 3D ground-models: City Rail Link, New Zealand <i>P.A. Kirk, T.J. Ireland</i>	71 – 77
9. AGS Data – Subsurface Data in Digital Format in Hong Kong <i>A.C.S. Lai, A.C.T. So, S.C. Mok</i>	78 – 83
10. The prevalence and detection of UXO from the wartime bombing of Hong Kong <i>P.S. Lee, J. Schultze, A. Mazur</i>	84 – 94
11. The Use of Floating Breakwater Technology in Hong Kong <i>Roger Lee, Jade Tang, Andy Yu, Gordon Ng</i>	95 – 105
12. Stability Improvement to a Loose Fill Slope in a Housing Development Project <i>Chester S.C. Leung, S.L. Chiu, Eric F.W. Lee</i>	106 – 111

13. Application of Real-Time Remote Monitoring on Flexible Barrier <i>Samson K.H. Leung, Geoffrey Pook, Petra M.Y. Lee, Anthea H.Y. Seto, Clayton Lei</i>	112 – 119
14. Adaptive updating of soil properties through monitoring data for improved prediction of excavation response <i>Horace M.K. Lo, Andy Y.F. Leung</i>	120 – 129
15. Connected Data – A digital journey in geotechnical engineering <i>Terence Lok Lun Lui</i>	130 – 135
16. The Use of Building Information Modelling <i>A.D. Mackay</i>	136 – 143
17. The Use of Underwater Cone Penetration Testing in Hong Kong <i>A.D. Mackay, Rachael Chan, Jesse Wong</i>	144 – 149
18. A New Approach to Modelling the Ground <i>Didi Y.Y. Mak, Flora W. Luk, Haydn H.K. Chan</i>	150 – 159
19. Construction 4.0: Risk Reduction and Productivity Improvement through the use of Expert Systems <i>A.S. Maxwell & J.F. Zhu</i>	160 – 170
20. Full scale testing of rebar and SFRC segmental lining using independent and wireless instrumentation systems <i>X. Monin, R.B. Storry, G. Steiner, A. Maxwell, J. Zhu, S. Sparrow</i>	171 – 182
21. Application of Advanced Software Programmes in a Mega-sized Railway Project Involving Substantial Embankment Construction <i>Sammy C.Y. Ng, Kelvin C.L. Tse, Ricky T.L. Lee, X.W. Miao</i>	183 – 189
22. Early Insight into Subsurface Risk: A Digital Platform Approach <i>R. Pot, A.M. Van Noort, I.J. Solomon</i>	190 – 199
23. Spatial Data Infrastructure of Ground Investigation Information <i>Minly M.L. So, Marvin Wirjomartono, Edward C.F. Lee, Haydn H.K. Chan</i>	200 – 205
24. Physical Modelling Study of a Debris Flow Impact on a Flexible Barrier <i>Daoyuan Tan, Jianhua Yin, Jieqiong Qin, Zhuohui Zhu</i>	206 – 214
25. ISO 19650 – A International Approach to the Journey of Industry Integration, Digitalization, and Innovation <i>Llewellyn Tang, Mark Bew, Ya Wen, Y.S. Lee</i>	215 – 225
26. Integrated Application of Shoring Modularization, Digitalization and Monitoring Sensors in Deep Excavation Project <i>Gavin S.H. Toh, C.N. Ho, S.A. Jenkins, K.L. Tsui, U.S. Lai</i>	226 – 235

27. Digitization of Geological Data for Clearer Understanding and Enhanced Productivity <i>Gavin S.H. Toh, Javier S.L. Yip, C.N. Ho</i>	236 – 243
28. How Big Data Benefits the Design of ELS in Hong Kong and the Prospects of Design Automation in the Future <i>Gavin S.H. Toh, Javier S.L. Yip, C.N. Ho, Terence L.L. Lui</i>	244 – 252
29. Digital Application in Foundation and ELS Designs <i>Gavin S.H. Toh, Martin C.K. Yip, Ivan Y.H. Kwok</i>	253 – 261
30. Digital Mapping of Discontinuities <i>Dennis Wong, Kitty Chan, Stuart Millis</i>	262 – 274
31. Innovations In Wireless Monitoring <i>K.L. Wong, T.S. Chan, Y.S. Cheung, C.M. Wong, J.A. Frame</i>	275 – 284
32. Building and testing virtual rock <i>L.N.Y. Wong, Y. Zhang</i>	285 – 293
33. Coupled analysis on optimizing installation configuration of energy dissipaters on flexible barriers <i>R.W Yu, Johnny C.Y. Cheuk, D. Song</i>	294 – 301
34. Automatic Digital Monitor for Standard Penetration Test in Ground Investigation in Hong Kong <i>Z.Q. Yue</i>	302 – 311

Some aspects of dynamic soil-structure interaction: excerpts from the 58th Rankine Lecture

N.J. O’Riordan

Arup, London

ABSTRACT

Geotechnical engineering is at its most unpredictable and uncomfortable when ‘live’ or dynamic loads on foundation systems are significantly higher than in the static or ‘dead’ load condition. Resilient infrastructure requires that the duration of bounce-back, the time to restore functionality after extreme events, is minimised. Codes attempt to deal with this using combinations of load and resistance factors that are sometimes drawn from fatigue or repetitive loading experience, however these can result in design solutions that either fail due to a lack of appreciation of the controlling parameters or that are uneconomic and overcautious.

The lecture examined dynamic soil structure interaction under extreme events: catastrophic collapse, wave loading, high speed rail track behaviour and urban seismic interaction using examples drawn from real-world installations. It showed how economy in design can be driven using unified soil-structure interaction modelling through advanced numerical analysis in which rate effects are explicitly considered. It provides a springboard from which resilient, performance-based design methods can be developed and improved, with the promise of feedback from instrumentation and digital data analytics.

A recording of the complete lecture is available online at <https://www.youtube.com/watch?v=44x4KuheZiQ>.

This paper contains excerpts from the lecture covering shear strain rate effects, catastrophic collapse, high speed rail track behaviour and urban seismic interaction.

1 Dynamic Soil-structure interaction

Dynamic soil-structure interaction can be examined using pseudo-static methods (Kausel 2010, Dobry, 2014) although the simplifying assumptions inherent in that approach, both in terms of structural mass participation and soil response to the associated loading in shear, will result in inefficient, and in extreme cases, insufficient foundation systems. The challenge is that extreme dynamic foundation loading situations are generally rare, involving statistically-derived forces with return periods that can be far in excess of 100 years. This means that full-scale testing of such foundations might involve field testing at inappropriate stress states, the mobilisation of very large lateral forces and moments, or limited to a small range of materials such as unstructured sands and kaolinite. Dynamic loading of appropriately scaled foundation systems can be simulated in the centrifuge (Muir-Wood, 2004, Kutter, 1992) however these present formidable challenges in terms of foundation construction simulation: for example ‘wished-in place’ test foundations in unstructured sands are relatively commonplace but do not adequately replicate reality. This can mean that the results from dynamic centrifuge tests can be dominated by the skill of the operators and the geometric limitations of the centrifuge box.

Main components of dynamic SSI

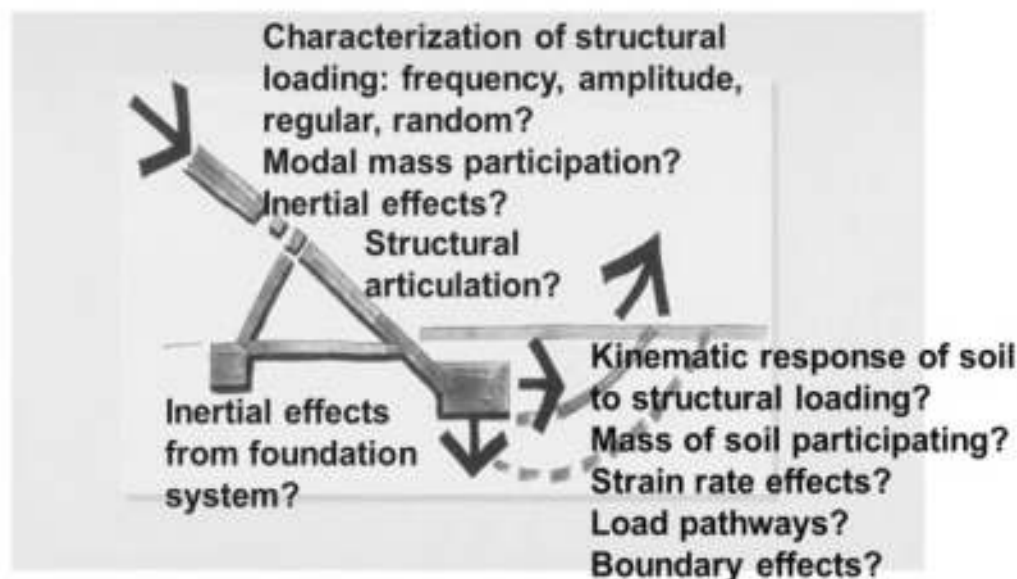


Figure 1: Dynamic soil-structure interaction context

Figure 1 shows the primary components of dynamic soil-structure interaction. Whilst the upper bounds of dynamic structural loading and approximate response can be established by assuming complete fixity at foundation level, the participation of soil in transmitting or responding to dynamic loading is more complex. Of importance is the mass of soil participating in the loading event: often the stiffness and strength of soils at comparatively high rates of strain are underestimated. This leads to premature formation of failure mechanisms, over-damping of soil response and under-estimation of the forces involved. It follows that the geotechnical engineer must attempt to model the ground as realistically as possible.

At this point, it is underlined that the results of numerical SSI illustrated in this paper are obtained using direct Finite Element (FE) methods (Bollisetti et al, 2015) using Arbitrary Lagrange-Eulerian formulation (LSTC, 2017). These methods facilitate the use of non-linear soil and structure models in extreme, large strain events. They give the greatest scope for economy in design and construction for situations where the governing design case is that for an extreme event.

O'Riordan and Almufti (2015), summarised the desirable attributes of 3D dynamic SSI analysis software as follows:

- Pressure- and rate-sensitive non-linear shear stress/strain soil properties
- Degrading G/G_{max} curves for the soil
- Water pressures: recognise volumetric & shear strain rates at excavation/construction and dynamic excitation stages
- Foundation systems
- Interface layers between structural elements and the soil mass that represent disturbed soil
- Representation of structures, vehicles etc: equivalent mass, stiffness and damping to capture dynamic behaviour
- Representation of construction installation sequencing
- Soil domain sufficiently large that boundary effects are negligible during dynamic loading
- Able to provide a 'rupture to rafters' (Ellison et al, 2017) process for seismic load cases

Currently, these attributes can potentially be met by LS Dyna (LSTC, 2017), ABAQUS (2013) and ANSYS (2016). Such formulations are comfortable with large strain calculations and discontinuities, for example when material boundaries separate. In a major review paper Bolisetti et al (2013) describe the advantages and disadvantages of a variety of software for dynamic analysis of soil-structure interaction analysis. Large strain problems can also be analysed using the Material Point Method (Fern et al, 2019) although at the time of writing the computational efficiency of the method over FEM is being quantified.

2 Strain rate effects, drainage and cyclic degradation

Increased foundation resistance at high rates of strain is well known and has influenced the preference for maintained load tests on piles rather than constant rate of penetration tests (for example Burland and Twine 1988). Such increased resistance can be effectively simulated using strain-rate dependent soil models.

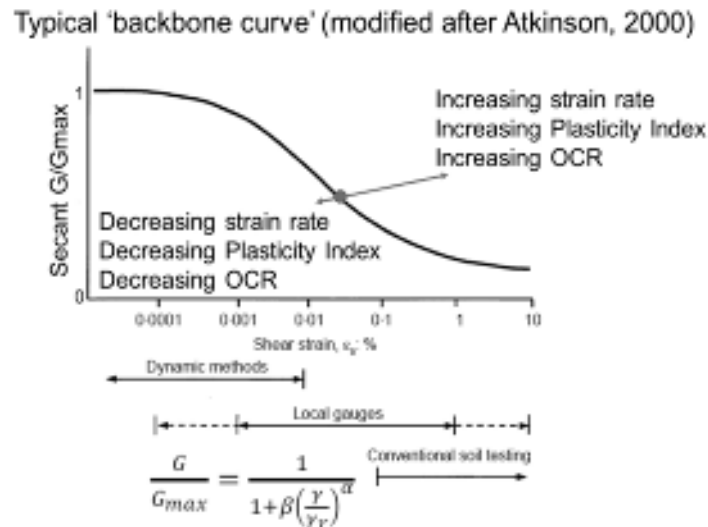


Figure 2: Backbone curve: effects of increasing strain rate, Plasticity Index and Overconsolidation

The basis for an integrated soil model is the G/G_{max} v shear strain degradation 'backbone' curve that originated with Seed and Idriss (1970) and has subsequently been developed by Darendeli (2001) and others. Figure 2 is redrawn after Atkinson (2000) and shows the leading components that control the location of the backbone curve along the shear strain axis and Figure 3 shows how unload/reload behaviour is often simulated using a format originally proposed by Masing (1926). Zhang et al (2005) demonstrated the large scatter in Masing damping for soils. Consequently the assumption of Masing behaviour should be critically examined for soil-structure interaction analysis, and to avoid over-damping soil response. Non-Masing behaviour can be examined through detailed examination of cyclic laboratory tests.

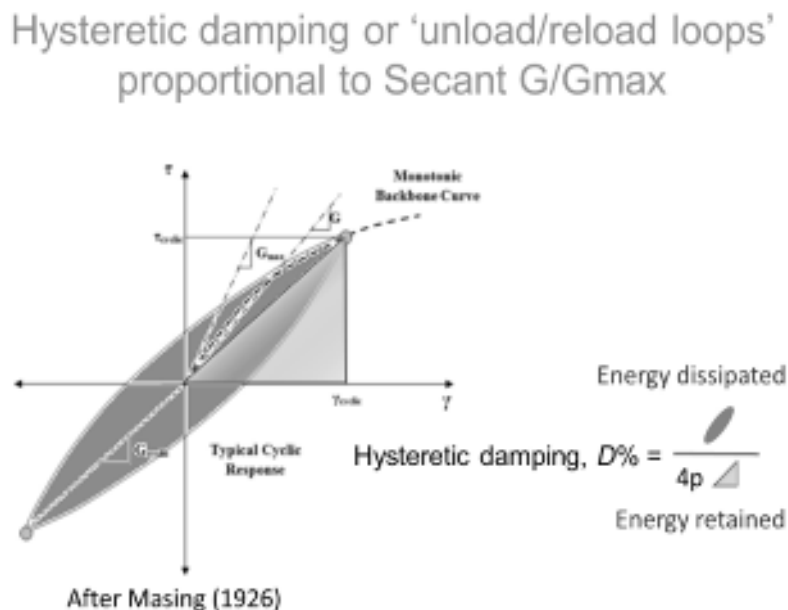


Figure 3: Backbone curve and definition of Masing(1926) Damping

O’Riordan and Almufti (2015) demonstrated a direct method for examining the effects of adjacent structures upon braced excavation in areas of high seismicity. They showed that excavation and seismic excitation stages could be represented sequentially so long as the backbone curve was adjusted for shear strain rate.

The effect of strain rate on stiffness and strength has been the subject of wide investigation. Many investigators have found excess pore pressure generation is suppressed as shear strain rate increases (Sheahan et al 1996, Biscontin and Pestana, 2001, Diaz-Rodriguez et al 2009) and show that the rate of increase in strength appears to be related to clay mineralogy. Thus low to medium plasticity clays might display a strength increase of about 5%/log cycle of shear strain rate and high plasticity clays closer to 20%. Figure 4 shows typical results for normalised strength variation and shear strain rate at these extremes.

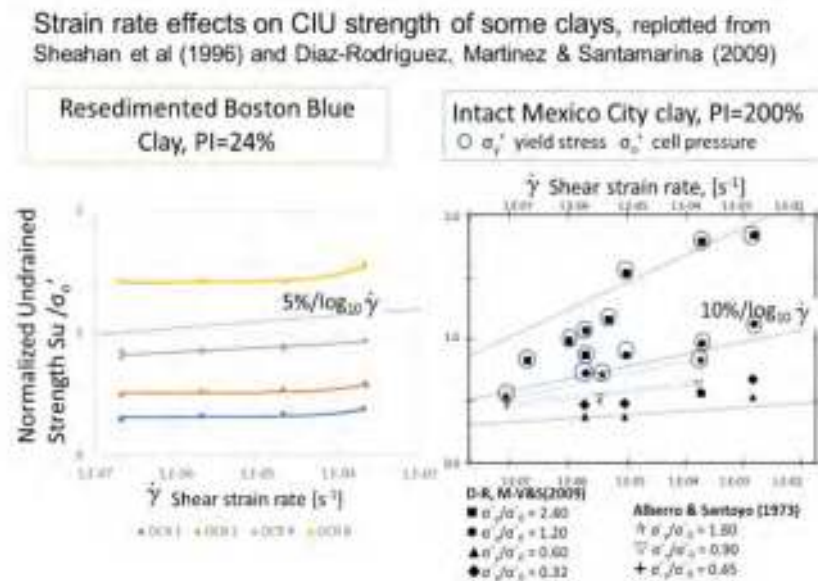


Figure 4: Effect of strain rate on stiffness and strength for two clays

Vardanega and Bolton (2013) examined a wide range of soils and concluded that, along with plasticity index (as a proxy for soil mineralogy) and overconsolidation ratio (OCR), measured secant shear moduli could be adjusted to a given shear strain rate using the following relationship:

$$G\dot{\gamma} = G_{\text{measured}}/Z$$

Where $G\dot{\gamma}$ is the shear modulus at a given shear strain rate $\dot{\gamma}$ and Z is a reduction factor dependent upon strain rate effect and the frequency of loading. For a clay-like material which displays an increase in secant shear modulus of 5% per tenfold increase in strain rate $\dot{\gamma}$, has a frequency of loading of 50Hz (typical loading frequency in a resonant column test), and shows the onset of plastic strains at 10^{-5} , Vardanega and Bolton (2013) found

$$Z = \left\{ 1 + 0.05 \log_{10} \left[\frac{100\pi(\gamma - 10^{-5})}{10^{-6}} \right] \right\}$$

This enables alignment of shear modulus degradation with increasing shear strain using undrained laboratory tests carried out at very different shear strain rates.

Typical shear strain rates for a triaxial compression test are at least 3 orders of magnitude slower than a resonant column test, as illustrated using tests on Vallerica Clay by Giorgiannou et al (1991).

Equally, the viscous behaviour of clays means that they are particularly susceptible to shear strain rate when approaching failure. Sheahan et al (1996) found that generation excess pore water pressures was suppressed at elevated strain rates and showed a typical increase in undrained shear strength of 5%/log cycle of shear strain in resedimented Boston Blue Clay. At higher PI's the effect is even more

pronounced and tests by Diaz-Rodriguez et al (2009) show up to 20% / log cycle of shear strain rate Figure 4. Figure 4 includes the effect of consolidating specimens at levels well above the Yield Stress (Burland 1990): the effects of such destructureation suppress rate effects and care should be given if soil properties obtained from such tests are to be used for design and analysis (Hight et al., 2007).

To effectively establish a priori whether the dynamic event is loading the soil next to the foundation sufficiently rapidly for undrained/zero volumetric strain conditions to pertain, closed-form solutions such as Gibson et al (1970) are extremely useful.

Gibson et al. (1970) uses linear elastic consolidation theory under plane strain and axi-symmetric loading conditions. They conveniently define a time factor T for dissipation of excess pore pressures such that $T = 2Gkt/\gamma_w b^2$.

Behaviour is effectively undrained if $T < 10^{-2}$. If the distance to a zero excess water pressure boundary is greater than the width of the loaded area, then using most plausible combinations of the leading parameters we find undrained behaviour can occur under these loading conditions

$> k=10^{-4}$ m/s cohesionless 'sand-like' soils experience a rate of loading/unloading < 1 sec. We can add that sand-like soils are dynamically not particularly strain rate sensitive.

$< k=10^{-9}$ m/s, cohesive 'clay-like' soils experience a rate of loading/unloading < 10 days. Clay-like soils are dynamically highly strain rate sensitive, particularly at low OCR and high PI.

A similar conclusion was reached by Zienkiewicz et al (1980). It follows that any use of Observational Methods, for example large excavations in clay-like soils (Nicholson et al, 1999) needs to explicitly consider the effects of drainage of stages in construction durations and the consequences of delays (and indeed acceleration) in construction rate.

3 Collapse and landslide modelling

Large strain FEM has been used for the inverse analysis of landslides and associated rockfall containment barriers (Cheung et al, 2018). The design of piled foundations in metastable slopes and in areas prone to karst collapse is particularly challenging. Muir-Wood (2004) considers the situation of piles embedded in an infinite slope where failure occurs along a defined plane, parallel to the ground surface. Treating the problem as pseudo-static and if the soil pressure is related to the relative movement between soil and pile, Muir-Wood was able to establish an exact solution albeit based upon empirically derived parameters. His analysis gives a value of lateral pressure close to the passive coefficient K_p when the relative movement between pile and soil is large.

Sartain et al (2009) considered failure of residuum around piled viaduct foundations using dynamic analysis and established that the maximum lateral pressure did not exceed $2 \cdot K_a$. The failing soil was unable to sustain stresses anywhere near the passive pressure of the soil. The derivation of parameters for the collapse analysis was based on back-analysis of multiple Menard pressuremeter tests carried out within boreholes formed using rotary airflush techniques. Fortunately records of karst collapse close to the viaduct location near the town of Centurion, South Africa, were well documented and evidence suggested that the time for the collapse to propagate from the base of the residuum at about 30m depth to ground surface was about 120 seconds, more or less the duration of a stress-controlled pressuremeter test. Figures 5 and 6 show that a reasonable fit to pressuremeter tests results can be found using a simple elastic-plastic model and stress dependent G values. Collapse scenarios can be readily simulated. The risk-based methodology used to establish credible scenarios of collapse in relation to viaduct foundation locations is given in Sartain et al (2011).

4 High Speed train loading and bending wave effects

There has been a certain amount of convergence in recent years between geotechnical and permanent way engineering (Li et al, 2015), largely as a result of operational desire to increase freight and passenger loading and increasing train speed on existing and new tracks. In many ways a ballasted track railway system represents an example of design to the Repairable Limit State (Honjo et al, 2010): the railway operator is accustomed to dealing with a certain expectation for ballast maintenance and track renewal as well as having contingency plans in the form of train speed restrictions (Schofield and Franklin,

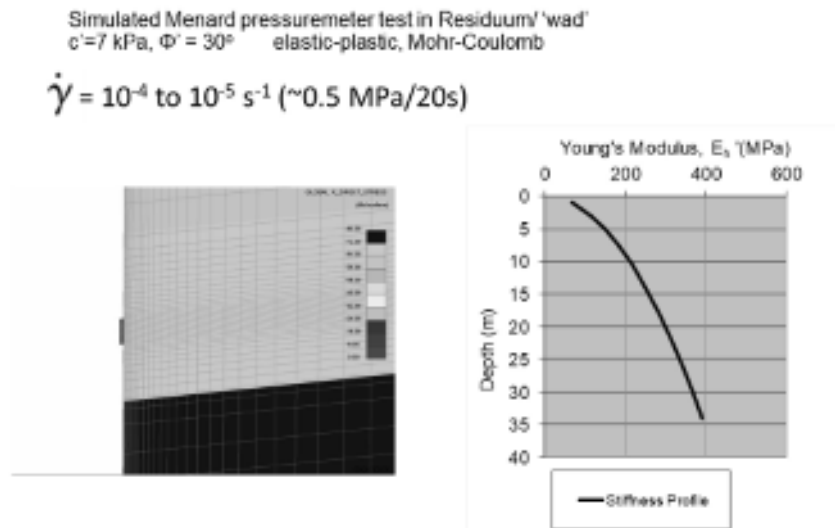


Figure 5: Pressuremeter test in silty sand Residuim

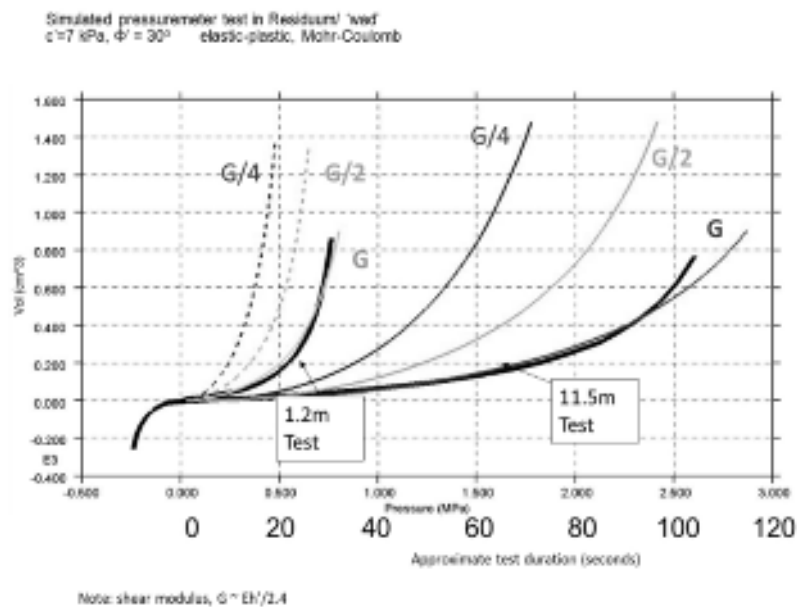


Figure 6: Calibration of stiffness to simulate pressuremeter test

2005) should track geometry become unacceptable. Urbanisation tends to drive increased train usage and the consequence of high maintenance, speed restrictions etc is becoming unacceptable both socially and economically.

Table 1: Design loading for fatigue analyses

	Axle load kN	Mean line load kN/m	Minimum axle spacing m	cycles/ day	cycles/ year	Maximum speed km/hr	weeks/ 100,000 cycles	months/ 1million cycles	years/ 10million cycles
Eurostar	170	21.5	3.0	3360	1226400	300	4	10	8
Commuter	130	22.5	2.5	2640	963600	160	5	12	10
UIC 71	250	80	1.6	-	-	-	-	-	-
Heavy Freight	225	80	1.8	720	262800	150	20	40	38

Table 1 shows the range of axle loading and numbers of cycles of loading that is required to be sustained by a track support system. Direct comparisons with microseismic activity, for example by Horleston et al (2013), show that the effect of the passage of a single train upon the support system can be considered to be equivalent to an earthquake event of at least magnitude $M_L = 1.5$.

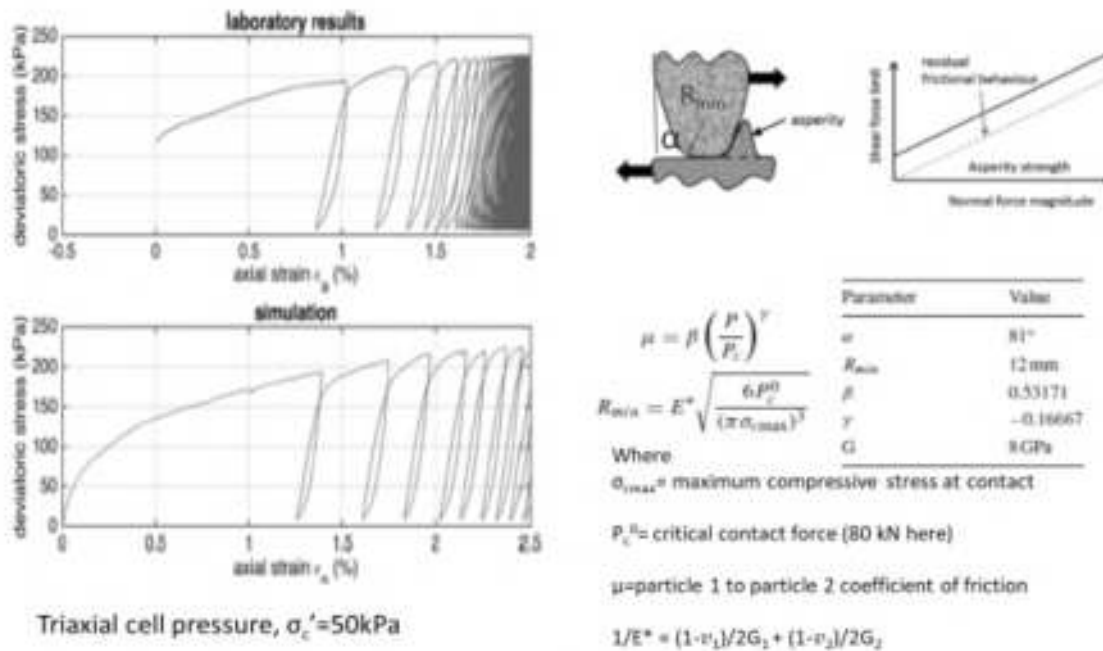


Figure 7: Damage-dependent strain DEM model for ballast (after Harkness et al, 2016)

Most existing track support systems comprise rails supported on individual sleepers or ties that rest upon unbound, angular ballast: a configuration that has changed very little in 2 centuries, despite a tenfold increase in train speed and axle load. Maintenance regimes have evolved around such systems. Figure 7 is taken from Harkness et al (2016) and shows the results of cyclic triaxial tests upon a 1/3rd scale model ballast. The progressive change in axial strain can be clearly seen and eventually the sample, after many cycles, reaches a point where the ballast behaves almost elastically: following a so-called resilient modulus that resembles that used in road and airfield pavement design (Brown and Selig, 1991). However the prediction of behaviour up to that point is both problematic and challenging; indeed the track may well have moved out of tolerance due to ballast attrition. Figure 8 shows the results of measurements of sleeper displacement under a Eurostar train on HS1 (O'Riordan and Phear, 2000) at Ashford (Priest and Powrie, 2009): the variable sleeper response is likely due to small variations in ballast behaviour at the sleeper interface. Milne et al (2015) show an extreme example of progressive ballast attrition that occurs where a sleeper coincides with a bridge joint.

Harkness et al (2015) and McDowell & Huiqi (2016) have achieved considerable success in using discrete element method (DEM) in order to quantify particle breakage and attrition. Such damage dependent strain modelling can be useful in describing structured soil behaviour under cyclic loading, as we shall see later.

Figure 9 shows the ratio of 'dynamic' track displacement to 'static' reported for track on soft ground on three sites as reported by Hunt (1993), Woldringh and New (1999) and Holm et al (2002). As can be seen, the ratio increases with increasing train speed. Pseudo-static analysis can mimic this behaviour, by trial and error, however 3D dynamic analysis can substantially improve predictive behaviour and enable optimisation of the track support system, as shown in Figure 27b.

In Figure 9 the track support system has been modelled using resilient moduli for the diverse materials involved and this considerably simplifies the prediction of track behaviour under increasing axle load. In particular, pseudo-static analysis can be used in order to examine ground vibration problems, especially those involving resonance when train speeds reach the bending wave velocity of the track support system and the Rayleigh wave velocity (Krylov, 2001).

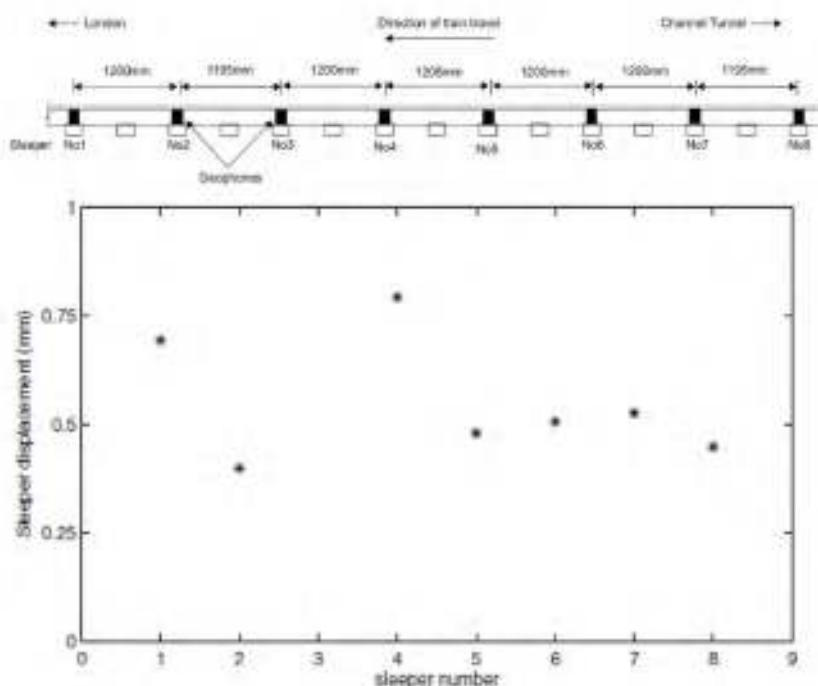


Figure 8: Measured sleeper displacements under Eurostar, 260 km/hr (72 m/s) at Ashford, Kent Priest & Powrie (2009)

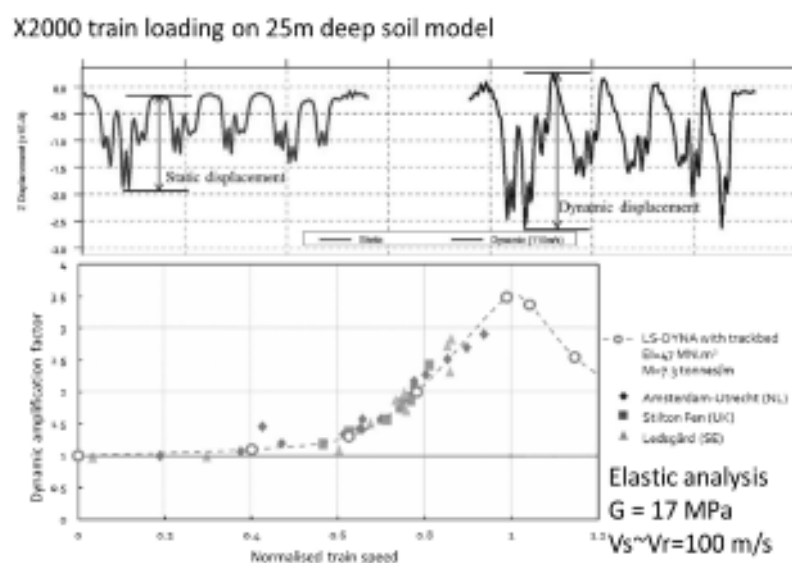


Figure 9: Elastic analysis of X2000 train loading on 25m deep soil model and comparison with selected case records

Bending wave velocity in such analyses can be given by

$$V_b = \sqrt[4]{\frac{4EI k_v}{m^2}}$$

where EI is the track system (simplified to an equivalent beam) bending stiffness, m is the mass of the track system per unit length and k_v is the vertical spring stiffness per unit length. EI for a typical well-constructed ballasted track support system is found to be approximately 47 MN/m² and this is associated with a mass of about 7 tonnes/m.

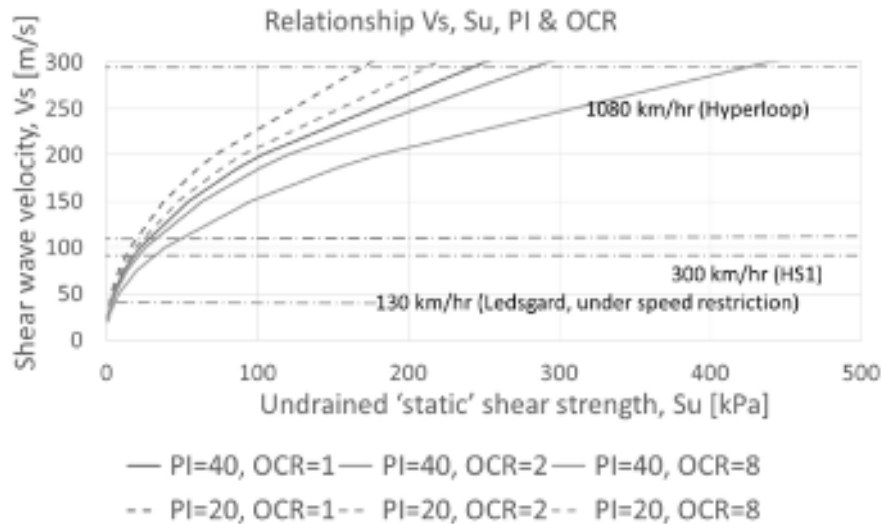


Figure 10: Unfactored screening threshold for clay-like soils under high and very high speed train loading

Gazetas (1991) shows that k_v is linked to the soil shear modulus by the approximation $k_v \approx \frac{1.2 G}{1 - \nu}$.

We also know that shear modulus can be related to shear wave velocity, which in turn is almost identical to the Rayleigh wave velocity (Richart, 1962). It follows that a simple screening process can be used to establish whether resonance or 'bow-wave' effects can be expected a priori, thereby enabling appropriate track support to be designed and thereby avoiding the large track displacements a posteriori, for example measured at Stilton Fen (Hunt, 1993) and Ledsgard (Holm et al, 2002). This is shown in Figure 10, using $G_{\max}/S_u$ relationships from Koutsoftas and Fisher (1980), Vardanega & Bolton (2013) and the author's datasets.

Non-linear soil model v. calibrated linear elastic model

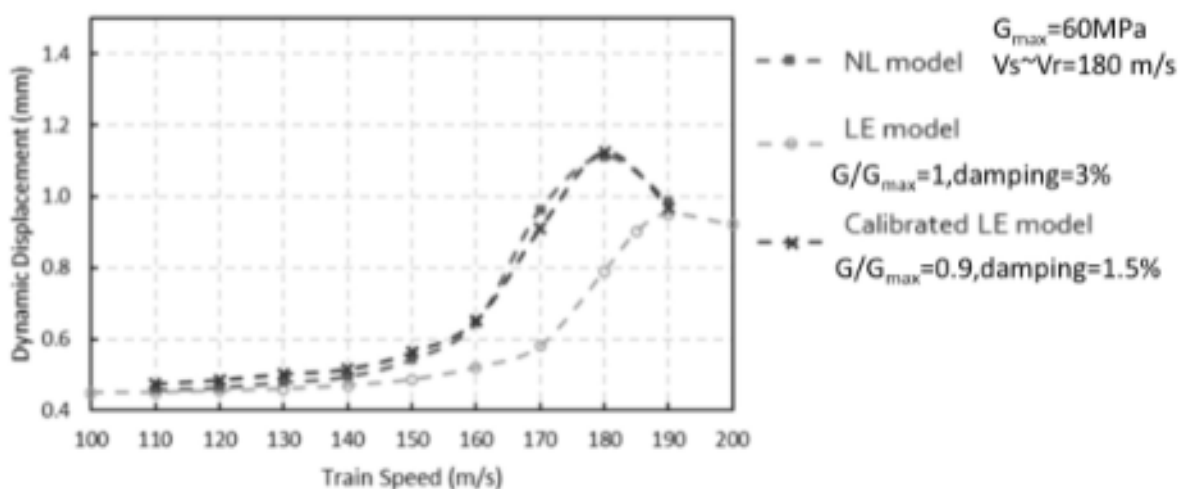


Figure 11: Non-linear and liner elastic simulation of dynamic amplification of train loading

Moving to full 3D, non-linear analysis for the examination of dynamic displacement amplification, Figure 11 shows the results for a train speed of 180 m/s, coinciding with an identical Rayleigh wave velocity and bending wave velocity for the track support system. An equivalent linear elastic soil

beneath the track support system can mimic non-linear dynamic behaviour but only through trial and error: importantly Figure 12 shows that an unadjusted, a linear elastic soil model, set at the same shear modulus as an equivalent non-linear model will tend to be over-damped and will underestimate dynamic amplification.

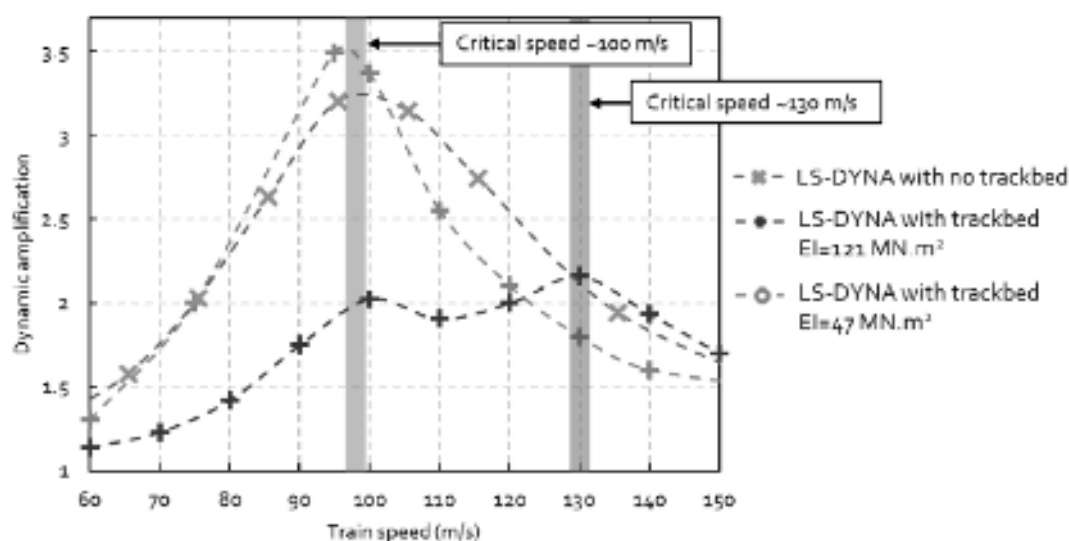


Figure 12: Stiffening effect of slab track on dynamic amplification of train loading

Figure 12 shows how an increased trackform bending stiffness, in this case a modest increase from 47 MNm² to 127 MNm², can reduce dynamic amplification.

It can be seen from the above that potential resonance problems at high and very high train speeds can be overcome and controlled by increasing the track support system bending wave velocity and increasing the bending stiffness. High speed rail of the future thus will benefit from stiffer trackslab support systems than the ballasted trackforms that are still prevalent in many high speed rail systems across the world.

5 Earthquake loading: urban interaction effects

Bolisetti & Whitaker (2015), in a major study of structure soil structure interaction (SSSI) analysis methods showed that significant restraint to foundation movement and ground loss/gapping can occur with an adjacent deep basement. The flipside of this is that the basement sees more load from the adjacent structure as confirmed by recent centrifuge testing of tall buildings adjacent to deep excavations and tunnel boxes by Hashash et al (2018).

O'Riordan & Almufti (2015) confirmed that in temporary works, equivalent static methods can substantially underestimate load effects from adjacent buildings and Ellison et al (2017) summarise the asymmetric loading that can occur in a deep basement inserted between two very tall towers in San Francisco. O'Riordan et al (2018) examine in detail processes for far-field site response analysis for performance-based design in near-fault situations where velocity pulses can dominate structural behaviour and those where soft soils can produce substantial magnification of bedrock motions at the ground surface. These far-field response analyses can be further refined using data from vertical arrays and form a reliable basis for examining urban effects and SSSI.

O'Riordan et al (2018) includes a back-analysis of the recent Puebla earthquake in Mexico City (19 September 2017).

Measurements across the lakebed during the recent Puebla earthquake highlight the potential for entire city blocks to interact and change the composition of the shaking of building foundations. Figure 13 shows the shear wave velocity profile across the Mexico Basin at locations west to east. The softest soils are found in the east, and Figure 14 shows the accelerometer recordings at the NAIM site. The duration of shaking is around 100 s. Figure 15 shows the measured values at the Aux Station within an urban area about 10 km from NAIM. It can be seen that the duration of shaking is not only significantly

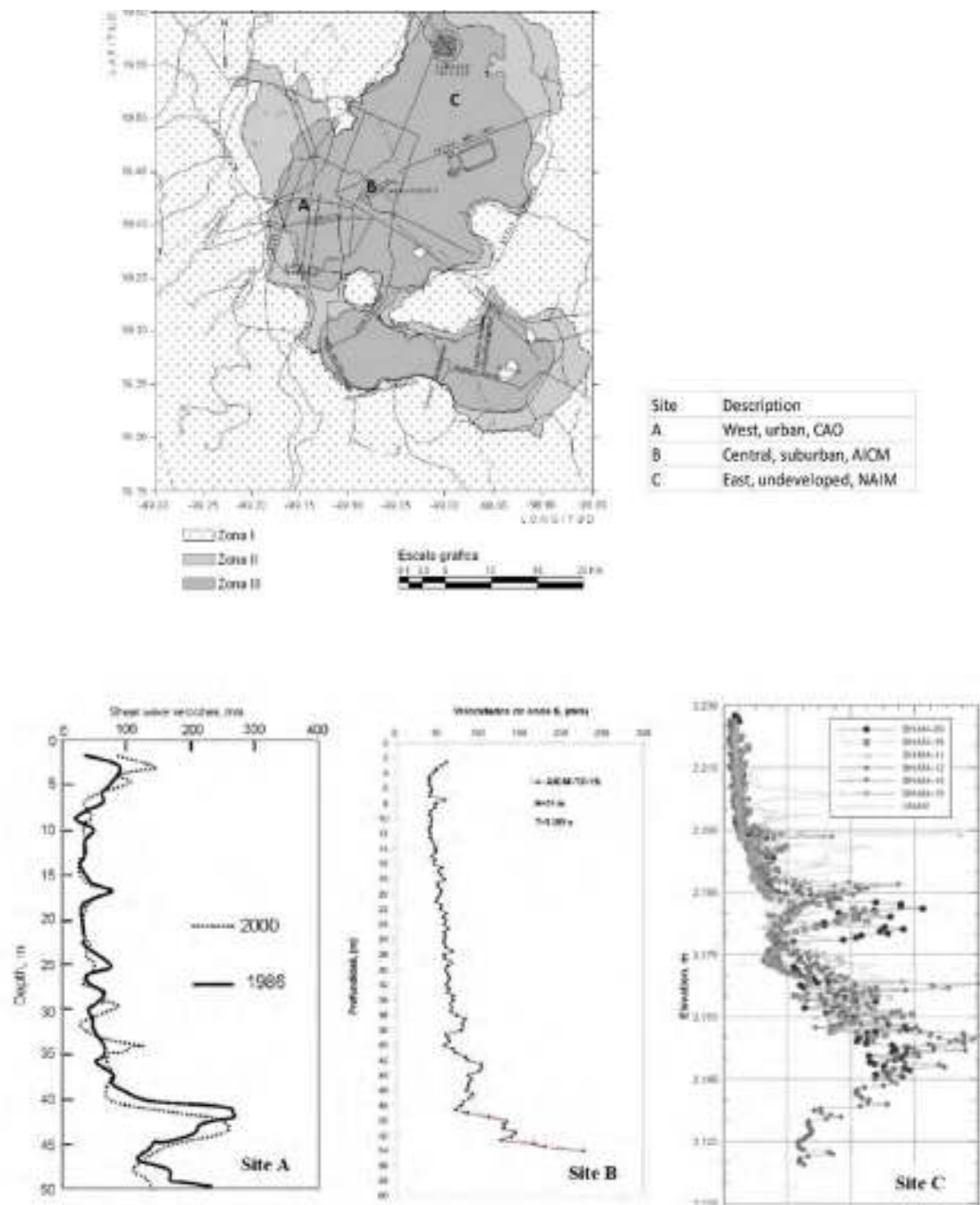


Figure 13: Variation of shear wave velocity across the lakebed zone, Mexico City

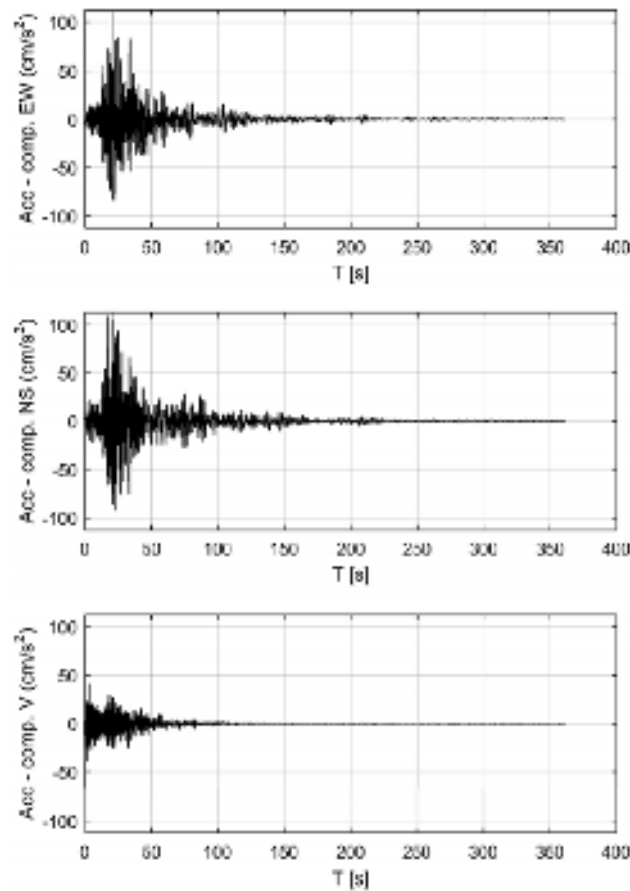
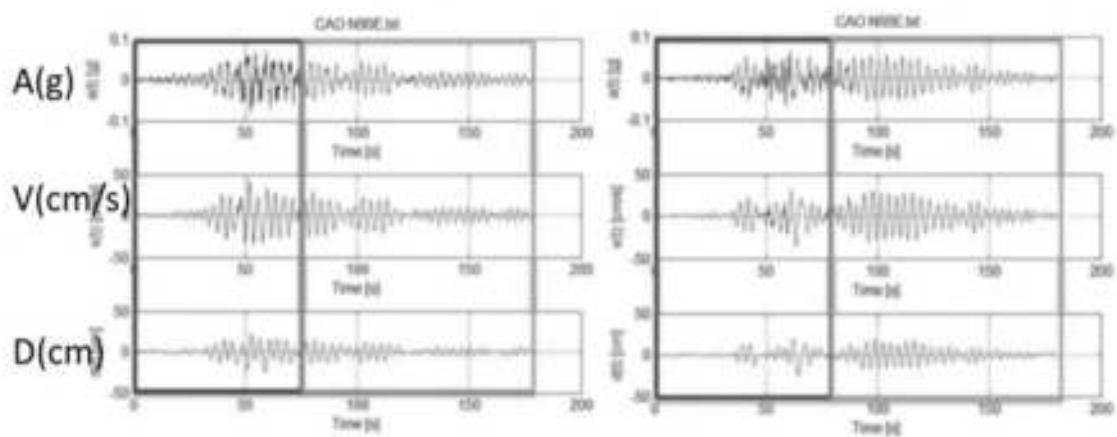


Figure 14: Triaxial accelerometer measurements at NAIM site, Puebla earthquake, 19.09.17

CAO Station, urban lakebed zone, Mexico City - Beating 2nd Phase



Michoacan EQ 19 September 1985

Figure 15: Triaxial accelerometer measurements and spectral analysis at Aux. recording station, Puebla earthquake, 19.09.17 (inset showing proximity of NAIM and Aux. station locations)

longer in the urban site, the record shows that a distinct second phase of shaking happens, beating at a period of about 2 to 3 seconds. The spectral amplification, and associated double peak acceleration spectra, that have been ascribed to 'basin effects' (Reinoso & Ordaz, 1999) may in the Mexico City case and other cities on soft soils in areas of high seismic activity be more likely due to urban, anthropogenic effects. Speculative analytical work by Guegen et al (2002), using observations in the Roma Norte district at the stiffer, western edge of the basin, suggested that urban interaction effects could be anticipated if the kinematic interaction ratio of the buildings to the underlying soil is higher than approximately 0.1. The calculated ratio for the urban area of Aux Station is 0.87, and thus kinematic interaction at a city block level is almost certain to have contributed to the measured movements and, to the west of the urban area, catastrophic damage of some buildings.

On stiffer soils and those prone to liquefaction, the potential for adjacent building damage due to structure-soil-structure interaction (SSSI) effects can also be large. Long term construction works adjacent to large existing structures are particularly vulnerable especially when the participating modal masses of the works are large and have widely differing period. Figure 16 is reproduced from O'Riordan & Almufti (2015) who considered the seismic exposure of a tall building to an adjacent multipropped excavation. Using earthquake simulations scaled to a return period of between 50 and 100 years, they demonstrated how seismic inertial loading from a tall building adjacent to a deep excavation with four levels of struts can deform the excavation support system sufficiently to produce a permanent increase in strut load. They found that strut forces can be nearly twice the force calculated for the slow excavation process itself.

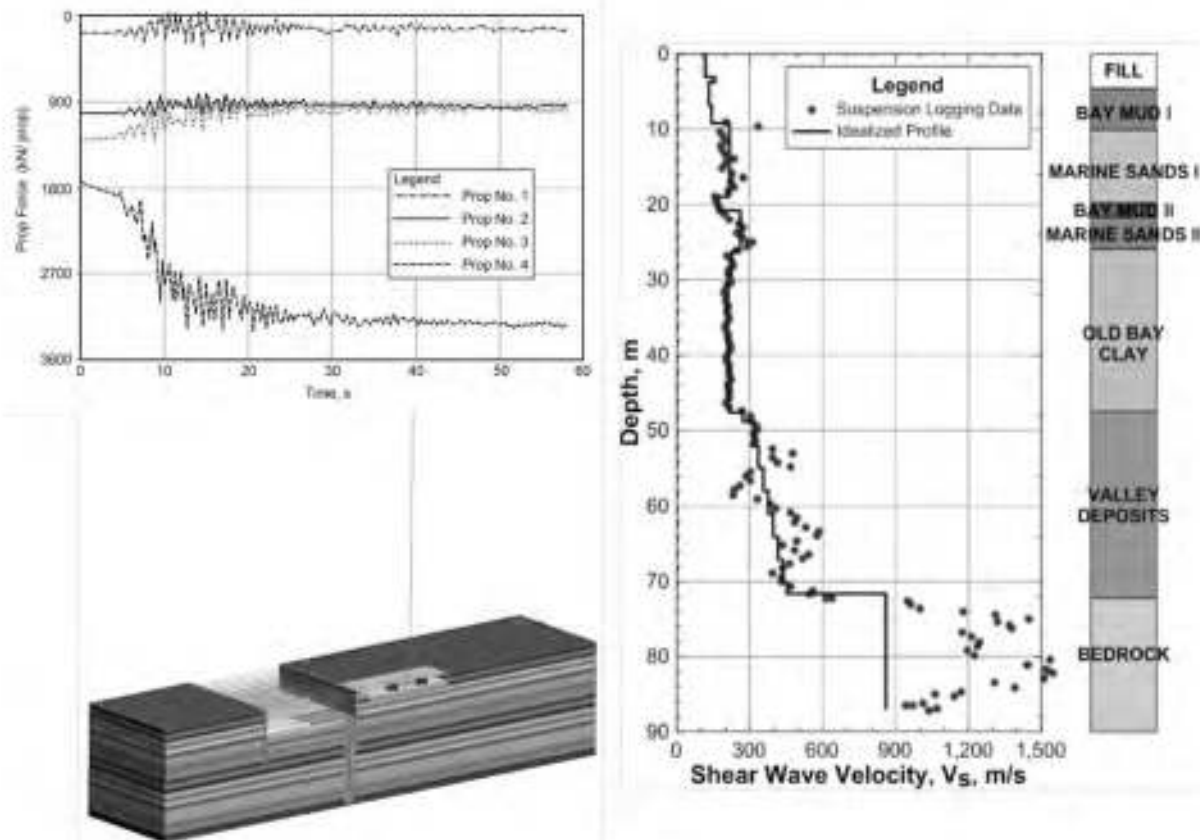


Figure 16: Seismic interaction of tall building and multi-strutted excavation (after O'Riordan & Almufti, 2015)

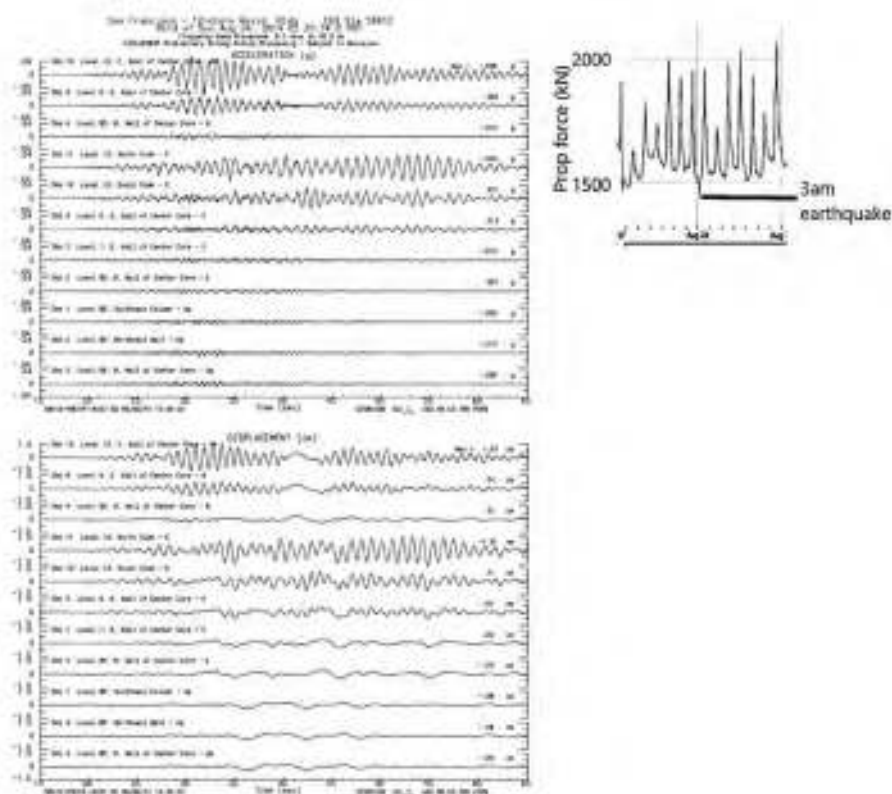


Figure 17: Measurements of load at lowest level of strut, Salesforce Transit Center, and comparison with measurements at adjacent 5 level basement. Napa earthquake 24.08.14.

Figure 17 contains results from the real-time monitoring for the Transbay Transit Center (McLandrich et al, 2013) showing the measurement of strut loads in a bottom level strut around the time of the Mw=6.0 Napa earthquake that took place at 3 a.m. on 24 August 2014. It can be seen that the repetitive daily change in force arising from changes in strut temperature is perturbed by a tiny drop in force at the time of the earthquake itself. Also in Figure 17 is the public record (CGS, 2014) of accelerometer and associated displacement time histories taken at 11 locations down an adjacent 12 story building with a 5 level, 18m deep basement. The records show an almost complete attenuation of the inertial response of the superstructure above level 1 by the multi level substructure (traces labelled B1, B2 etc). The response of the strut monitoring system is compatible with this measured behaviour of an adjacent, multi-basement structure.

SSSI effects can also strongly influence the design of buried structures in the permanent condition involving shaking from earthquakes of much longer return periods than those considered for the 'temporary' works. O'Riordan et al (2018) examine how appropriate site response analyses are required before embarking upon detailed SSI.

Ellison et al (2017) consider the effects of two very tall structures, Salesforce Tower (326m tall) and 181 Fremont (244m tall), located either side of the 20m deep, 60m wide and 500m long Transbay Transit Center rail station box in the middle of San Francisco downtown. These tall towers have foundations socketted into bedrock at about 70 to 80m depth and the Transit Center box is essentially groundbearing, equipped with 1800 tie-downs to deal with buoyancy effects in some parts of the base slab under long term rising groundwater pressures. The first mode period of the Transit Centre is approximately 1.2 sec, as summarised in Figure 18. Modal mass participation of all adjacent structures was considered, mostly using the simplified methods given in ASCE 7-16. Particular attention was given to the taller surrounding buildings and Figure 20 38 illustrates the modal shapes associated with fixed base shaking of 181 Fremont in the horizontal directions together with the associated participating building mass. The vast majority of the mass participates in the first two modes at a between 6.4 and 6.9 sec.

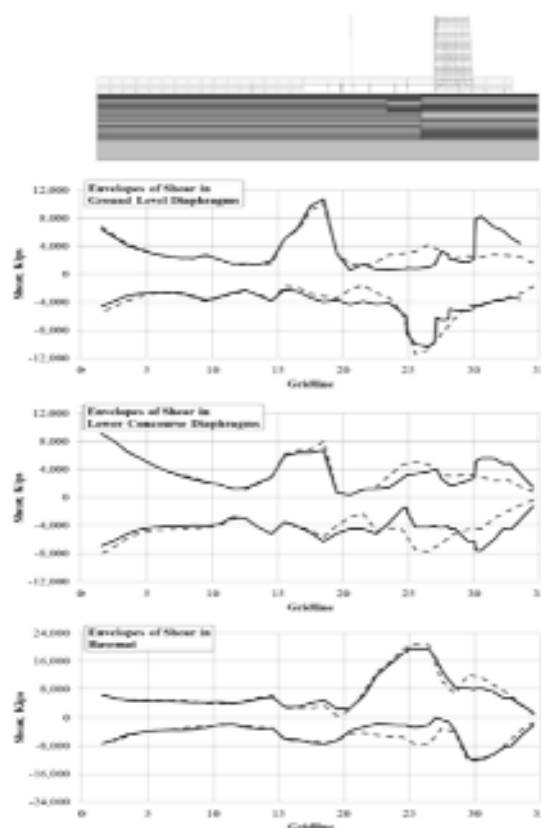


Figure 20: Salesforce Transit Center: shear force envelopes in ground, mezzanine and base slabs from interaction with Salesforce Tower, with and without 181 Fremont tower. 975 year return period earthquake

Large scale numerical analyses such as these currently require the processing power of multiple, linked computers and can take a day to reach a conclusion. Nevertheless by systematically creating versatile soil models of the type described in this paper, together with close attention to construction sequence and structural modelling, appropriately optimised, economic design solutions can be found to these complex dense urban situations.

The physical modelling of SSSI is complex and there have been few successful, published simulations of the effects. Until there are more case histories of instrumented building and tunnel response from real earthquakes, the most promising area of research appears to be in the use of seismic centrifuges. There are constraints in the way in which construction sequencing, tall building inertial effects etc can be simulated in the centrifuge. Using dry Nevada Sand as the soil medium, city blocks on shallow foundations have been modelled (Jones 2013) and successfully simulated numerically by Bolisetti and Whittaker (2015) Figure 21 shows one of the test arrangements whereby a 10 storey structure is placed in varying configurations adjacent to low rise structures. For this situation on a medium dense sand there were measurable interactions but of comparatively low significance. This was not the case for the centrifuge tests on tall buildings and adjacent buried tunnel structure, again using dry Nevada Sand, reported by Musgrove et al (2017). Lateral stresses substantially higher than initial K_0 stresses were measured in the tunnel wall, and the researchers report successful application of numerical analysis of the centrifuge tests using LSDyna (Hashash et al, 2018).

Summary and Conclusions

- Increasing urbanisation has changed the nature of geotechnical engineering
- Everything is connected, and we can ‘instrument the Anthropocene’, beyond mere construction monitoring
- A resilient future will require much greater feedback from performance of foundation systems
- Dynamic numerical analysis can be used to calibrate soil models against extreme events

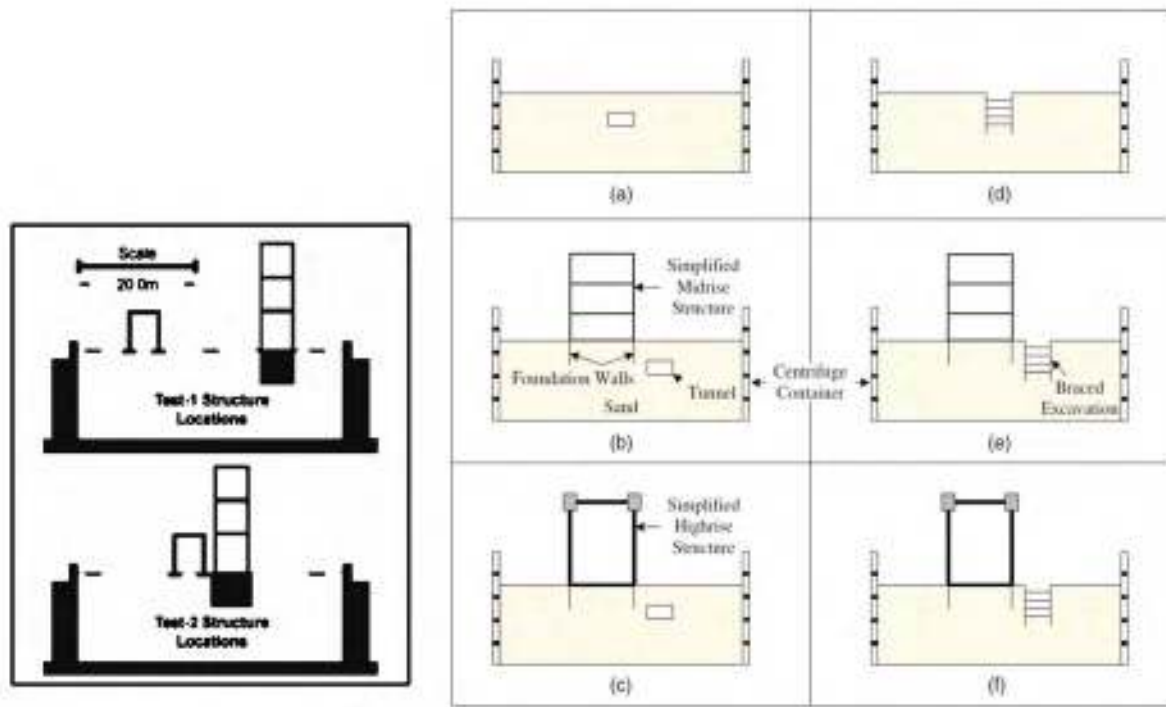


Figure 21: Examples of adjacent building interaction studies in the seismic centrifuge
(after Jones, 2013, and Musgrove et al, 2017)

- These analyses can explicitly include the changes in rate of loading of the ground during sudden, extreme events as well as that in slow, normal service loading.
- Transient 'extreme' in-situ tests such as the CPT and Pressuremeter, as well as output from vertical arrays of seismic accelerometers can be successfully analysed using advanced numerical analysis
- Unified soil models and increasing computing power enable progress towards performance-based design and resilience
- We can look forward to increasing feedback from long term instrumentation systems: 'Big geotechnical data'
- We now have the tools to articulate to stakeholders the consequences of extreme events on foundation systems

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Application of Digital Engineering to Geotechnical Tunnel Design

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ABSTRACT

With Building Information Modelling (BIM) now mandatory for many civil engineering projects, there is an increase in emphasis and awareness of technological advancement and the term digital engineering has come to the horizon of engineers. While BIM focuses on building/structural modelling, digital engineering has a broader sense of application for wider disciplines. Digital engineering enables better use of geotechnical data and facilitates innovative design.

In this paper, examples of the application of digital engineering during the detailed design process of a recent tunneling project is presented through data acquisition, processing and design stages. The paper discusses how digital ground investigation (GI) data in AGS format was imported and managed systematically through a Geographic Information System (GIS) geodatabase system. Multi-disciplinary data such as geotechnical parameters, published geological materials and available information from other nearby projects is consolidated into a single geodatabase to facilitate interpretation. The use of digital data assists engineering geologists and geotechnical engineers to interpret and construct a comprehensive 3D ground model efficiently. The use of photogrammetry for example, is not new to geotechnical engineering, but digital engineering methods today facilitate the way the data is handled and processed which makes photogrammetry an even more valuable tool in the field of rock engineering. Finally, Discrete Fracture Network (DFN) was used to characterize rock mass condition in critical design areas. The DFN provides a good indication of rock mass behaviour and provides the essential statistical information about the potential formation of rock wedges which helps to inform the design process.

1 INTRODUCTION

A precise and informative geological model is critical to the design of linear infrastructure projects such as tunnels. The process of developing a comprehensive ground model requires efficient handling of a great deal of data. Recent developments in digital engineering introduce new forms of data acquisition and management and this data can now be brought together to create 3D ground models in much more efficient and accurate ways than has not been possible in the past for the benefit of engineering interpretation and design.

This paper is based upon a case study of the application of digital engineering techniques to the temporary support design of a major tunnel project. The process of extraction and processing of digital ground investigation (GI) data from AGS file using Geographic Information System (GIS) is also discussed. The benefit of adopting the GIS approach is that data can be processed very efficiently, and it can be used

to facilitate the creation of 3D geological models. A demonstration of the use of photogrammetry as a technique to acquire high quality geological data is also presented. This includes drone photogrammetry of open rock slope and stationed camera techniques inside a tunnel during construction. In critical areas with specific design concerns, the use of Discrete Fracture Network (DFN) model with digital mapping data to build a representative rock mass is also described.

2 PROJECT SETTINGS

The examples presented in this paper are based on the temporary support design works for a hard rock tunnel excavated using the drill and blast method. Besides the main tunnels, there are several short branch tunnels located in the vicinity of formed rock cut slopes. These tunnels are located in crystalline rock of granite, and volcanic tuff with basaltic intrusions. Based on the published geological maps, local geological features such as faults and contacts between granite and tuff are anticipated to intercept the tunnel alignment.

3 DIGITAL ENGINEERING APPLICATION TO DESIGN STAGES

3.1 Use of Digital GI data

An understanding of the subsurface condition is crucial to high risk geotechnical engineering projects. Our understanding of the ground is ideally established through comprehensive GI campaigns before the start of the project. In some large-scale engineering projects the amount of GI data can be considerable, which requires large resources to handle and more importantly make use of this data which is essential to design and construction success.

On this example project, more than 1500 boreholes which include 213 project specific boreholes are located within the site boundary. Among the project specific boreholes, more than 20 types of field tests and laboratory tests were carried out. In order to extract and make best use of the information from the extensive amount of GI records efficiently, AGS data was used as the major source.

AGS data format has been implemented in Hong Kong since 1993 and is a standard which specifies how GI and laboratory test (LT) data are stored digitally. Most of the information contained in the fieldwork reports and the laboratory testing reports are transferred into the AGS files. The AGS data can be broadly divided into two major types; numeric data and text data. Numeric data includes data from coordinates of GI stations, sampling/coring depths, measurements on rock cores, test results from in situ field testing and laboratory testing to instrumentation monitoring data. Text data refers to data which is stored in the form of a text string, such as the name of the GI station, description of geologic material, weathering grade of rock, etc. Information related to the building of engineering geological models for tunnel design can be directly extracted from the AGS files. These include RQD, weathering grades, defect conditions, packer test results and laboratory test results from UCS, elastic moduli, and rock joint direct shear tests. The test data listed is provided by the GI contractor usually in the form of an Excel spreadsheet, and this data forms the fundamental basis of rock mass characterization.

3.2 Adopting Geodatabase approach in data management

Apart from the base GI data, the development of an engineering geological model also requires topographic data, site survey data, published geological information, geological mapping and probing data from different digital source formats. A geodatabase was employed to consolidate and manage all sorts of information within one single database. A geographic information system (GIS) platform was selected as being the most appropriate because the interpretation of geological conditions require complex spatial operations and interpolation of geological data as well as visualization of data in various ways.

Different types of data are imported into the geodatabase as feature classes, including point, polyline, polygon and multi-patch class. While the classes define the geometry of the data, within the feature classes themselves is more detailed information stored in a tabular format. For example, a borehole may be visually represented as a stick log, but once the user clicks on the stick log all relevant information is

displayed via a pop-out table. In the geodatabase this data can be either graphically selected or filtered based on user defined attributes. This provides significant advantage over other data management systems because it is fast and intuitive to use, and avoids the need to search through data tables. In addition, once the geodatabase is set up, all members of the design team can interrogate the same data set. This ensures everyone is always working with the latest information because it is automatically updated.

Visualization is another important feature of the GIS. The same set of spatial data can be visualized at different styles according to the data attributes. For example, the same set of GI data can be visualized as colour sticks representing the RQD value, weathering grade or geological unit, or as graded symbols to represent the UCS values. Data from different formats, such as CAD drawings, georeferenced maps or digital terrain models can also be visualized in the same graphic system using a layered structure. The visualization helps to identify possible locations of geological features and define different geological horizons with similar properties.

Another benefit of adopting the geodatabase approach is that new information can be readily introduced into the database which can be constantly updated with additional information. Having all the data located in a single database improves data review efficiency and makes it much easier to use. Figure 1 presents an example of incorporating probing results and monitoring data into the same geodatabase. Here, the probing results can be directly compared to the existing ground model data to check whether the predicted ground conditions are still valid. Better interpretation of monitoring results can be achieved by comparing the monitoring result with the ground model simultaneously.

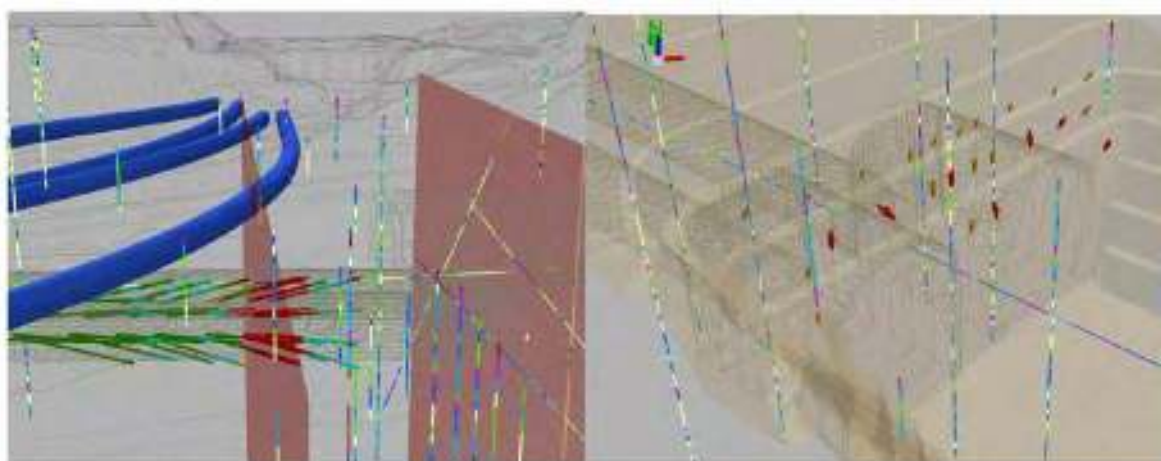


Figure 1: As-built probing results incorporated into the geological model (Left).
As-built monitoring data (arrows) incorporated in the ground model (Right)

3.3 3D geological/ground model

At the planning stage, the application of 3D models can facilitate planning of a cavern / tunnel development (GEO, 2018). In the example project the data sets included both vertical and inclined boreholes as well as horizontal directional coring (HDC). The 3D geological model was developed to resolve the problem of projection of GI data at different orientations, and this provided improved spatial linkage to geological features and changes in rock mass conditions (Thum et al., 2015). The model was constructed by combining the digital GI data, digital topographic data, site survey data, published geological information and interpretation from aerial photographs. A screenshot of the geological model is presented in Figure 2. The information from the vertical boreholes, inclined boreholes and HDC are presented as colour stick logs at the corresponding locations against the proposed tunnel alignment (in purple), the existing tunnels (blue) and inflow data from another project to form the 3D model. This facilitates the interpretation of the geological features / fault zone (flamingo colour) along the tunnel alignment as shown in Figure 2.

The 3D model provides improved interpretation of the spatial relationship between geological features and tunnel alignment. This helps to provide a much more reliable and accurate presentation

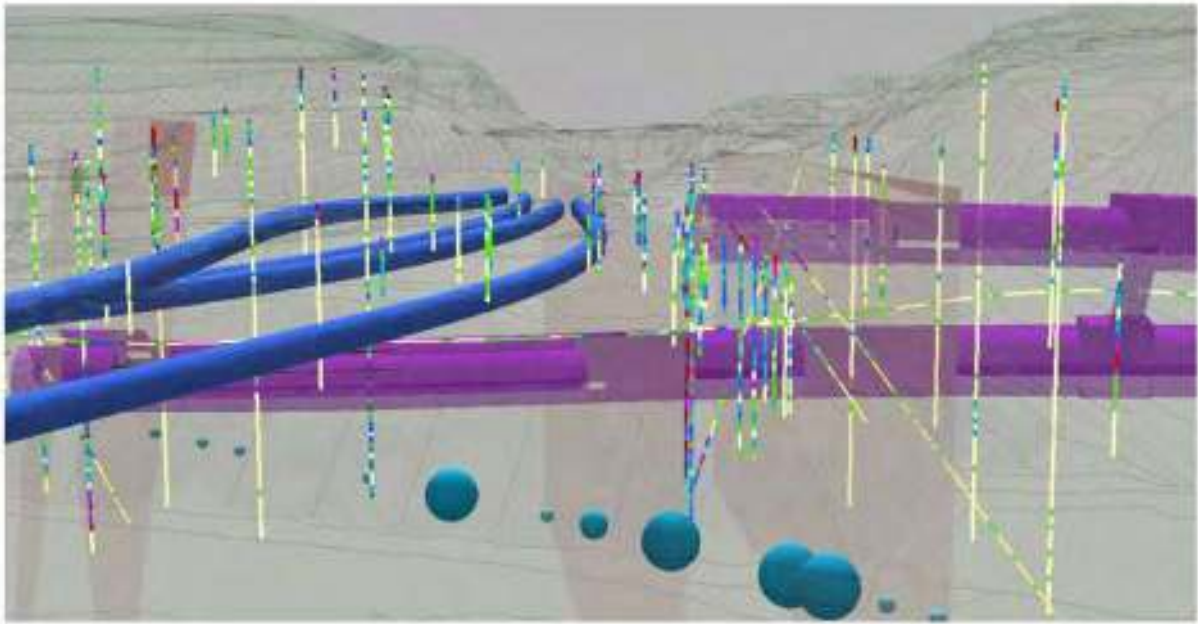


Figure 2: Example of 3D Geological Model for a tunnel project

of important geological features with potential risks associated with any adverse ground conditions which may be encountered. The benefit is that the models can be rotated manually so that he/she can interrogate specific zones of interest without having to try to understand the model through 2D cross sections.

3.4 Application of photogrammetry survey in data acquisition

Photogrammetry is not a new discipline but digital engineering has enhanced its applications. The survey makes use of a pair of stereo-pair photographs to generate 3D survey data points. The stereo-pairs can be acquired from either drone or stationed cameras on the ground. The advantage of applying photogrammetry over other surveying methods in geotechnical engineering is that an orthomosaic image can be created when building the digital terrain model (DTM). The orthomosaic image allows detailed examination of surface conditions and terrain feature measurement carried out over the DTM. In this example project, photogrammetry was employed to provide topographic profile, tunnel excavation profile and digital mapping data for design input.

Figure 3 presents an example of the DTM generated from drone photogrammetry at a portion of the site. The survey accuracy from the drone photogrammetry was typically around 100mm. This is sufficient for most excavation purposes. Regular repeat survey enables the designer to trace and monitor the excavation profile and even identify evolving displacements which could indicate the onset of instability. The orthomosaic image provides visualization of features at the site, records of works progress and site activities at different stages, and visual imagery of locations of difficult access. These are valuable tools for geological interpretation, works planning, site progress monitoring, and monitoring against trigger levels.

A trial of digital mapping using the technique of photogrammetry was also carried out in the main tunnels. The setup involved a stationed camera with purpose mounted lighting and ground control point. Figure 4 presents an example of the DTM generated inside the tunnel, the DTM covers the tunnel face and perimeter. The maximum accuracy attained with the stationed camera is 3-5mm, which is sufficient for rock joint mapping and surveying. Figure 5 shows the digital mapping of rock joints, excavation profile comparison and shotcrete thickness analysis carried out with the software package ADAM Tech 3DM Analyst. The 3DM Analyst was then compared to actual site data. The results from the digital mapping and shotcrete thickness analysis were compared to field mapping records and coring of the shotcrete at the site. The comparison shows that results from photogrammetry are in very good agreement with the site data, and in terms of the comparison with field mapping, the photogrammetry captures more relevant information. What is just as impressive is that the images can be captured very quickly. Our



Figure 3: An example of DTM and orthomosaic image created by drone photogrammetry

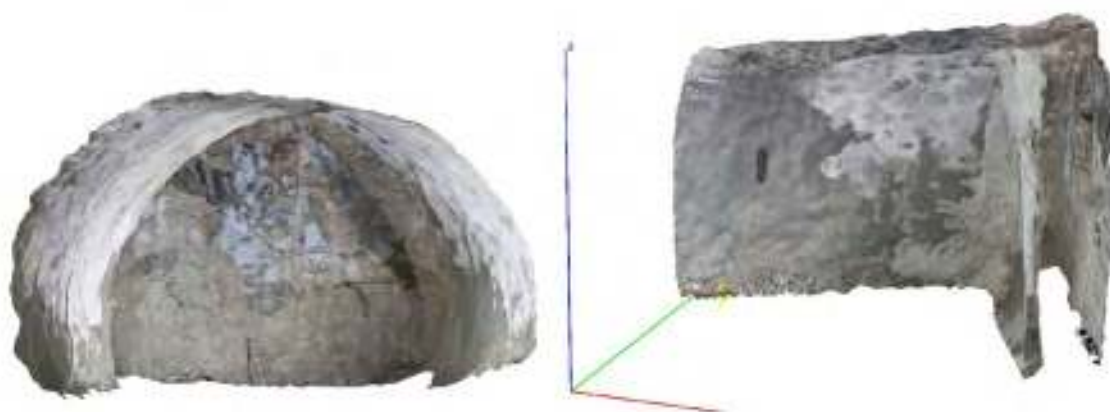


Figure 4 Example of DTM and orthomosaic image of tunnel excavation created by stationed camera photogrammetry (Left). Comparison of two consecutive excavation faces inside the tunnel (Right)

trials proved that the set up and photography can be completed within about 15-20 minutes at the face, minimizing disruption to tunneling operations, and indeed the amount of time geologists need to be at the excavation face. The photogrammetry technique can also be applied for tunnel deformation monitoring with the setup of ground targets. (Scaioni et al. 2014)

For applications which do not require high levels of precision, the photographs can be acquired by drone which can cover a very wide range in a short space of time.

There is particular application for photogrammetry in large caverns where excavations are typically mapped and excavated in discrete stages. Digital engineering processing enables defects to be accurately tracked and presented, reducing the chances of features being unrecognized as the excavation evolves. In terms of the large excavations required to form caverns, this aspect should not be understated. Images can be effectively stitched together so that an accurate geological model evolves allowing us the capacity to confirm conditions, predict the likely extent of problematic ground, and optimize support ahead of construction. Overall it is an improved risk management tool.

3.5 Application of discrete fracture network (DFN) method

During the detailed design stage, part of the newly formed rock cut slope was exposed and mapped. In order to better utilize the valuable mapping data, the use of Discrete Fracture Network (DFN) method with digitalized rock joint data was adopted as the primary review method to assess the properties of the jointed rock mass.

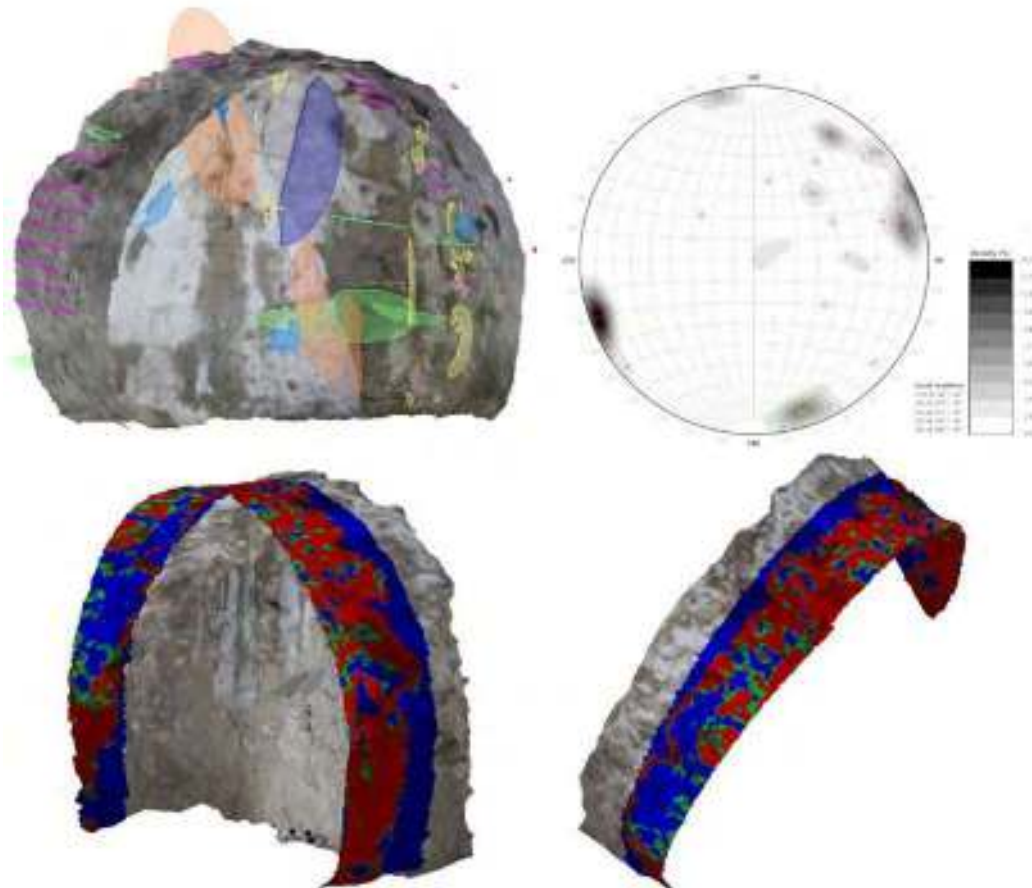


Figure 5: Example of digital mapping of rock joints in the DTM and the associated stereoplot of the mapped joints (Top Left and Right). Shotcrete thickness analysis carried out by ADAM Tech 3DM Analyst (Red <25mm, Green 25-50mm, Blue >50mm) (Bottom Left and Right)

In this example, the DFN model was created using Golder's proprietary FracMan software. Since only part of the exposed slope was available for mapping, the fractures in the unexposed part are extrapolated from the available data. Each rock joint in the mapping record is digitalized as 3D trace lines, Figure 6. The traces are exported to FracMan and subdivided into different data sets according to the measured

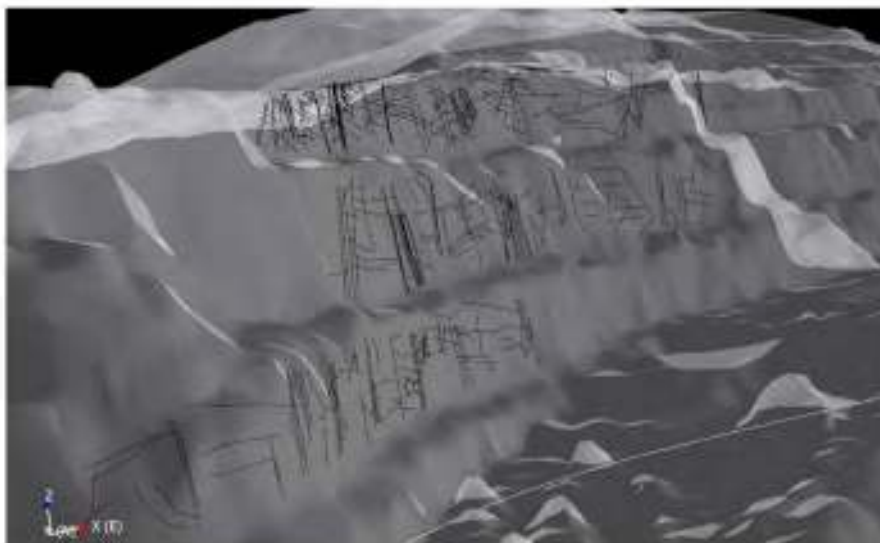


Figure 6: Screenshot showing the imported 3D traces of from mapping record, ground profile produced by photogrammetry survey

orientation/dip angle. The trace sets are then reproduced in FracMan using the bootstrap method with an initial intensity value. A trace intersection is created to compare the fracture sets generated with the intensity value to traces from the mapping records. These steps are repeated until an intensity value matches the traces. This validation process is illustrated in Figure 7. After the validation, a fracture model for the entire slope is generated as in Figure 8.

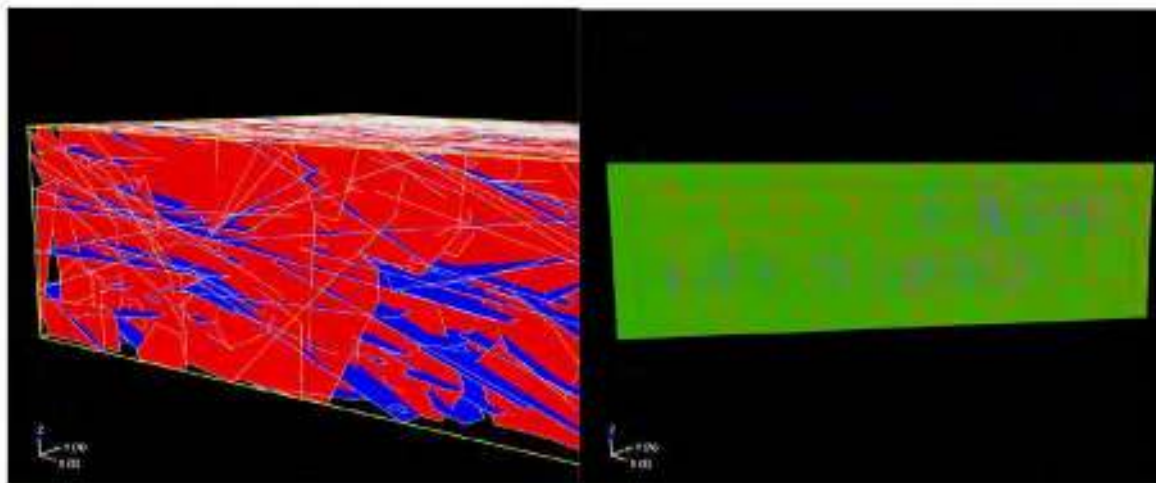


Figure 7: Fracture model generated using initial intensity value (Left).
Validation with traces mapping record (Right)

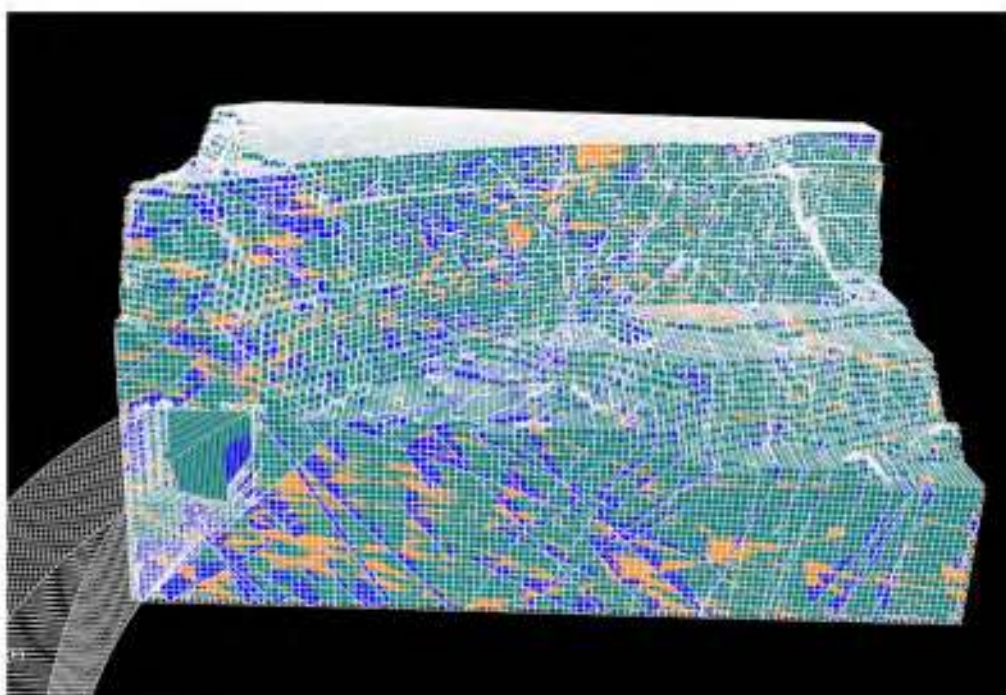


Figure 8: The DFN generated for the entire slope after validation process (with FE grid)

The DFN is then used for two major assessments, (i) the evaluation of rock mass parameter for geomechanical simulation in the anisotropic rock mass (Elmo et al., 2014) and (ii) the anticipated wedge size and stability (Rogers et al., 2017). Prior to the stress evaluation, a finite element (FE) grid was generated and the DFN properties were upscaled to the grid to create a geomechanics grid. In this case the stiffness tensor for the grid is upscaled according to the modulus of the intact rock and the stiffness of the fracture. The output stiffness tensor for the grid cell corresponds to the concentration of fractures in the grid cell. Figure 9 presents the output of the fracture upscaling which shows that grid cells intercepting fractures have a much lower stiffness when compared to cells that have not

intercepted fractures. The stress change in the geomechanics grid is then analyzed by FracMan built-in FracStress Geomechanics function. The FracStress analysis uses a FE model with nodes corresponding to the vertices of grid cell and the properties in the grid cell (stiffness tensor in this case) to calculate the displacement, strain and stress in each cell. The result of FracStress analysis is presented in Figure 10. It is observed that the high principle stress is concentrated in areas with arching and/or with high stiffness, which matches the predictions.

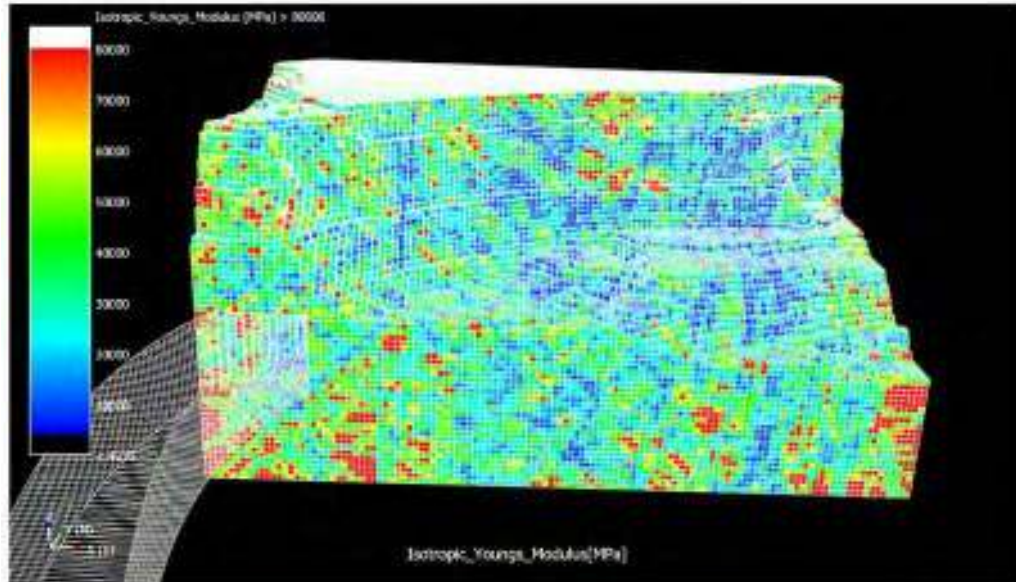


Figure 9: Stiffness tensor FE grid for the entire slope as output of the fracture upscaling

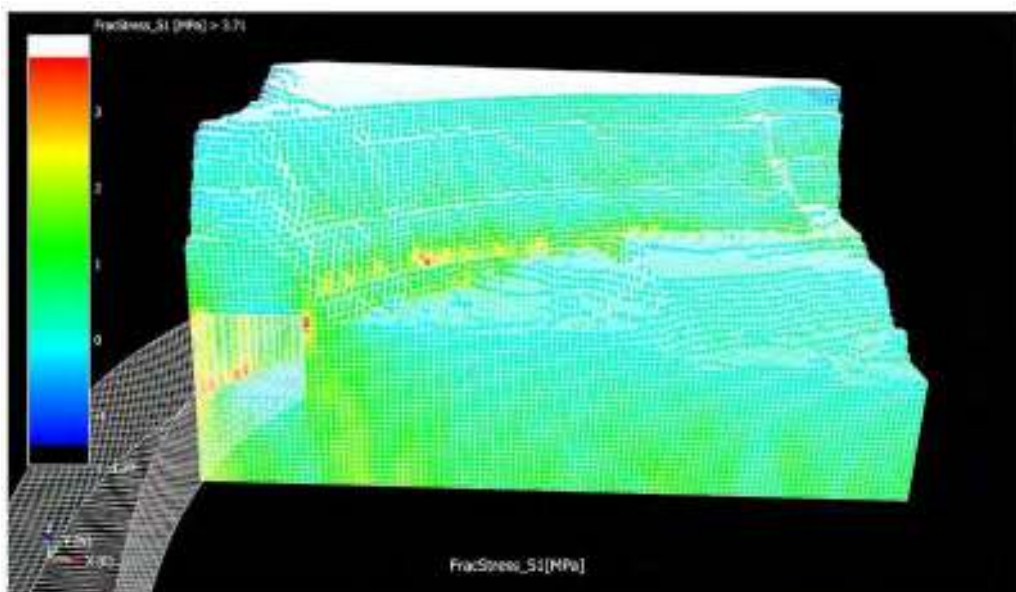


Figure 10: Result of Stress Change induced by the tunneling using FracStress analysis

Apart from stress analysis, DFN can also be used for wedge size realization and preliminary kinematic review. In FracMan Rock Wedge analysis, rock blocks are identified from the interior of tunnel/slope surface with intersecting fractures. Trace maps are formed on the tunnel/slope surface and the fracture surface until all the faces of a rock block are defined. The mass and the volume of the rock block is then calculated and used in stability analysis. Figure 11 presents the result from the Rock Wedge analysis, the analysis provides a 3D visualization of the possible sizes of the associated rock blocks. Kinematically unstable rock blocks are indicated in red. Information about each individual block, such as volume, weight and geometry, can be viewed in tabular format. This allows more accurate estimations of the likely block sizes which forms the basis for design.

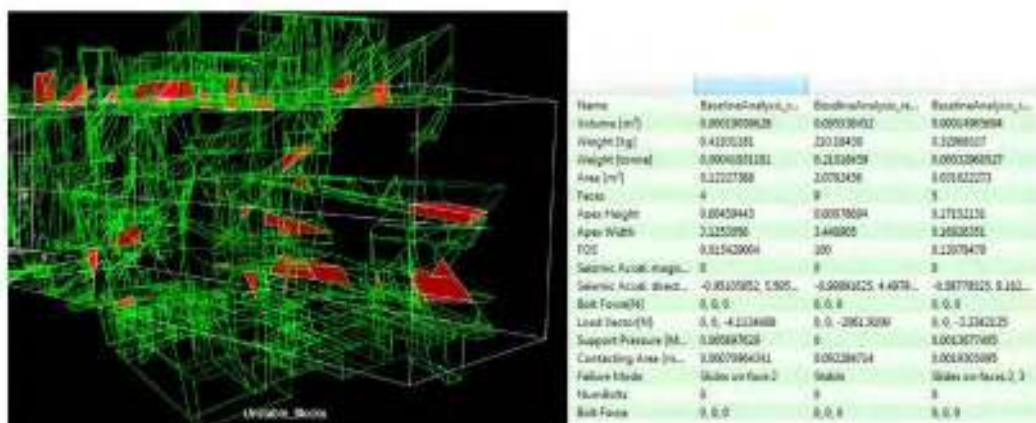


Figure 11: 3D visualization of rock blocks from Rock Wedge with interrogated analysis of individual blocks

The results from the DFN analyses provide a better realization of the anticipated ground conditions. The FracStress geomechanic grid can be exported to other numerical software such as FLAC 3D or 3Dec for detailed design analysis. The result from Rock Wedge analysis can be used for further statistical and kinematic calculations.

4 SUMMARY

This paper presents examples of the application of digital engineering to rock engineering. The use of AGS data reduces the time required to extract useful information from a large amount of GI data. Consolidating different sources of data into a single geodatabase enhances the efficiency in data management, interpolation and geological interpretation. The application of photogrammetry in the generation of digital terrain models, digital mapping and design verification provides the means for high quality data acquisition. For critical locations in the tunnel design, the use of digital mapping with DFN provides a more realistic representation of the ground conditions with enhanced understanding of the rock mass characteristics.

The key summary point is that with the skilled application of digital engineering to complement existing tools, we can make better use of available information in the interpretation and design of engineering projects and so this has immediate effect of increased efficiency, accuracy and not least of all, safer design solutions.

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The Slope Greening by Tree Planting on a Soil Cement Fill Slope in Hong Kong

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ABSTRACT

Many residential developments in Hong Kong involve site formation works at hillside terrains where fill slopes are often formed as a result. There has been an engineering practice that the fill is required to be compacted to achieve a minimum degree of relative compaction of 95% (Ng & Lumb 1980). It is not uncommon that fill material would be mixed with some cement as an additive to enhance the shear strength of the fill. On one hand, the cement additive helps increase the stability of the slope by means of the increased strength (Mitchell 1976). However, it makes the ground less favourable for vegetation growth due to establishment of a high pH environment (Mitchell 1981) in the soil cement mixture. In this paper, reference is made to a housing development project where attempt has been made to carry out slope greening by planting of trees and shrubs on a compacted soil cement fill slope. A set of design parameters, a list of local tree species, and the growth rates of tree seedlings had been obtained from a pilot study carried out earlier on with the project as reported elsewhere (Hau et al. 2018). As a follow-up on the work by Hau et al. (2018), this paper presents the result of the implementation of the key parameters for the slope greening work and illustration of the growth rates of the plants with time after plantation. A discussion and conclusion on the slope greening work to the soil cement fill slopes is given at the end of the paper.

1 INTRODUCTION

This paper addresses the challenges and measures adopted in carrying out slope greening works in a housing development project. The landscape design for this slope attempted a pilot study on site trial of tree planting on a soil cement fill slope (Hau et al. 2018). 700 tree seedlings in 20 species were selected and planted for the trial on a soil cement fill slope. The study started with plantation of the tree seedlings in Oct 2015 and they were maintained over an observation period for almost 3 years. The result of the pilot study provided a set of parameters for design of the slope greening in this project. Some principal findings from the study and details in slope greening design will be highlighted and discussed in this paper.

2 THE SOIL CEMENT FILL SLOPE FOR PILOT STUDY

The pilot study was conducted on a soil cement fill slope. It was a high cut slope and the profile of which consisted of a top layer of compacted soil cement fill of an average depth of 2.5m overlying slightly-to-moderately decomposed granitic rock (Figure 1). The soil cement fill was employed to replace the existing fill on slope which contained largely coarse gravels with occasional cobble sized moderately strong rock fragments that made the top soil difficult to be compacted, also barren and unsuitable for slope greening.

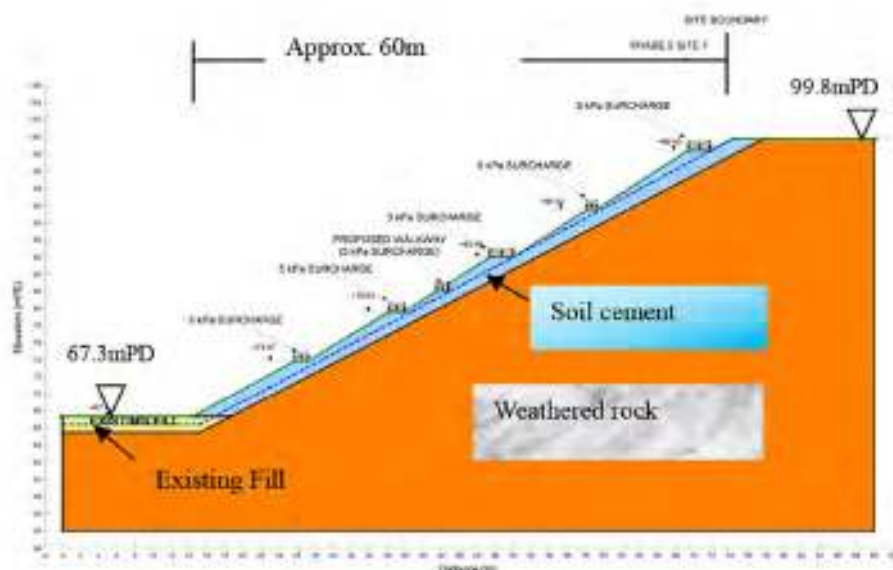


Figure 1: The geological profile of the trial area



Figure 2: Site plan of trial site - 3 tree planting plots on a compacted soil cement fill slope amid high rise buildings

The trial area measured approximately 700m² comprising 3 plots on the slope, namely Plot 1 to Plot 3 (Figure 2). The mix of soil cement with imported soil of Completely Decomposed Granite (CDG) and Portland cement was designed as shown below in Table 1.

Table 1: Composition of soil-cement mix (by mass)

	Portland Cement	Sand	Inorganic soil
Proportions	1	3	12
Specification	BS-EN197-1	BS1200	Free of organic matter and not more than 30% of soil particles passing a 63µm BS test sieve

The cement and CDG fill were firstly dry-mixed on site and placed by backhoe excavator, and then compacted in layers with mechanical vibrator, to a relative degree of compaction $\geq 95\%$ of its maximum dry density determined by compaction test to Geospec 3 (GEO 2001).

3 THE PILOT STUDY

A list of native species of trees and shrubs were identified from literature survey of local studies and engineering records. The pilot study aimed to find out from those native species of trees and shrubs which are the most suitable for growth on compacted soil cement fill slopes in this project. The selected species had to live with a hostile eco-environment, viz in soil with a high pH value (Mitchell 1981), reduced porosity and impinged biological activities due to the presence of cement additive (Hau et al. 2018). The trial area was in a shaded area where penetration of sun light is limited by the surrounding buildings above and below the slope. Seven hundred seedlings were planted, viz 35 seedlings for each of the 20 species of native trees and shrubs. They are listed in Table 2. Among them, 11 species were shortlisted from the review of previous studies and works that were cited in Hau et al. (2018); they were found suitable for growth on soil cement fill with a high survival rate at the end of observation periods. The other 9 species (marked with a plus sign in Table 2) were adopted in the study for the first time, being native species.

3.1 Planting design and results of monitoring

At the nursery, measurement of the stem height for each seedling had been taken and recorded as the baseline for growth determination in the study. The mean values of seedling-height of the trees and shrubs are given in Table 2.

Table 2: Species and sizes of seedlings planted in Oct 2015

Scientific name (Chinese name), code	Mean seedling height (cm)
<i>Carallia brachiata</i> (竹節樹), [Cb ⁺]	30.6
<i>Cyclobalanopsis championii</i> (嶺南青岡), [Cc]	33.8
<i>Cyclobalanopsis myrsinifolia</i> (小葉青岡), [Cm*]	24.3
<i>Cyclobalanopsis neglecta</i> (竹葉青岡), [Cn]	36.0
<i>Daphniphyllum calycinum</i> (牛耳楓), [Dc ⁺]	39.4
<i>Diospyros vaccinioides</i> (小果柿), [Dv ⁺]	33.4
<i>Elaeocarpus sylvestris</i> (山杜英), [Es**]	33.9
<i>Garcinia oblongifolia</i> (嶺南山竹子), [Go*]	52.3
<i>Ilex viridis</i> (綠冬青), [Iv]	41.1
<i>Lithocarpus glaber</i> (柯), [Lg ⁺]	57.7
<i>Litsea rotundifolia</i> var. <i>oblongifolia</i> (豺皮樟), [Lr*]	19.9
<i>Machilus chekiangensis</i> (浙江潤楠), [Mc ⁺]	34.5
<i>Phyllanthus emblica</i> (餘甘子), [Pe]	32.3
<i>Psychotria asiatica</i> (九節), [Pa]	32.0
<i>Rhaphiolepis indica</i> (車輪梅), [Ri]	68.4
<i>Rhus succedanea</i> (木蠟樹), [Rs]	42.3
<i>Schefflera heptaphylla</i> (鵝掌柴), [Sch]	18.8
<i>Syzygium hancei</i> (韓氏蒲桃/紅鱗蒲桃), [Syh ⁺]	56.5
<i>Viburnum odoratissimum</i> (珊瑚樹), [Vo ⁺]	37.4
<i>Vitex quinata</i> (山牡荊), [Vq ⁺]	17.3

+ Species adopted in the study for the first time

* Species had only 34 seedlings available

The 700 seedlings were planted randomly on the 3 tree planting plots in the lower two batters of the slope. The planting took a staggered pattern at 1m intervals vertically and horizontally, done from 5 to 16 October 2015. There was no other particular treatment to the seedlings after planting than watering and basic maintenance work following the guidelines in the General Specification (GS) for Civil

Engineering Works Vol. 1 (CEDD 2006). According to the GS, seedlings should be planted in wet season between March and September. The planting work of this trial, however, was done in early October 2015, slightly outside the recommended planting time of the year. This was not considered having significant effect on the growth of the plants as 2015 was not a year particularly dry, and given that watering and maintenance work was carried out under close monitoring during the site trial period. Over an observation period from October 2015 to August 2018, all plants were practically watered as necessary to avoid going wilted in dry season.

The first field monitoring was conducted on 12 November 2015, roughly one month after the completion of the planting work. The second field monitoring was conducted about 15 months later on 28 April 2016, at a time toward the end of the dry season. The latest visit was paid in the 33rd month of the planting in August 2018 at the end of the study.

During the first monitoring, it was noted that only 699 instead of 700 seedlings were actually found planted in. Of them, 3 were found dead. As a result, 4 species (marked with an asterisk in Table 2) had only 34 seedlings available; thus, only 696 seedlings were counted in the planting trial. The seedling survival rates in the first month ranged from 62.9% to 100% (Table 3). The overall mean survival rate was 97%, viz that the overall transplantation loss was only 3%. In the second monitoring on 28 April 2016, the seedling survival rates after the first winter dry season were still good for most of the species with a survival rate over 80%. The latest visit was paid at the end of 33th month of the planation. It was noted that, the survival rates dropped from an average of 97% to about 85% (Table 3). The mortality rate in the first dry season was considered high for some species, e.g., *Cyclobalanopsis neglecta* 62.9% to 51.4%. Besides, there were some seedlings that had been found killed and damaged by the construction activities.

Table 3: Seedling survival rates monitored at the 3 visits and Mean RHI of the seedlings at the 15th month (RHI15) and 33rd month (RHI33)

Scientific name (Chinese name), code	Survival rates in %			Mean RHI15 (April 2016)	Mean RHI33 (August 2018)
	2015	2016	2018		
<i>Carallia brachiata</i> (竹節樹), [Cb ⁺]	100	77.1	65.7	0.2708	1.1732
<i>Cyclobalanopsis championii</i> (嶺南青岡), [Cc]	100	80	77.1	0.3209	1.2888
<i>Cyclobalanopsis myrsinifolia</i> (小葉青岡), [Cm [*]]	97.1	91.4	88.6	0.7181	1.5430
<i>Cyclobalanopsis neglecta</i> (竹葉青岡), [Cn]	62.9	51.4	40.0	0.3189	1.3207
<i>Daphniphyllum calycinum</i> (牛耳楓), [Dc ⁺]	100	88.6	85.7	0.3302	1.4011
<i>Diospyros vaccinioides</i> (小果柿), [Dv ⁺]	100	91.4	77.1	0.1697	0.8998
<i>Elaeocarpus sylvestris</i> (山杜英), [Es ^{**}]	94.1	71.4	25.7	0.3547	1.0876
<i>Garcinia oblongifolia</i> (嶺南山竹子), [Go [*]]	100	94.3	80.0	0.1576	1.1291
<i>Ilex viridis</i> (綠冬青), [Iv]	100	85.7	68.6	0.2777	1.2569
<i>Lithocarpus glaber</i> (柯), [Lg ⁺]	100	88.6	88.6	0.3046	1.3729
<i>Litsea rotundifolia</i> var. <i>oblongifolia</i> (豺皮樟), [Lr [*]]	100	82.9	51.4	0.2293	0.9364
<i>Machilus chekiangensis</i> (浙江潤楠), [Mc ⁺]	100	91.4	57.1	0.2139	1.0723
<i>Phyllanthus emblica</i> (餘甘子), [Pe]	100	94.3	91.4	0.4081	1.3660
<i>Psychotria asiatica</i> (九節), [Pa]	100	77.1	22.9	0.0996	0.3679
<i>Raphiolepis indica</i> (車輪梅), [Ri]	100	88.6	91.4	0.2258	1.2863
<i>Rhus succedanea</i> (木蠟樹), [Rs]	97.1	82.9	77.1	0.1329	0.9120
<i>Schefflera heptaphylla</i> (鵝掌柴), [Sch]	97.1	71.4	57.1	0.2456	1.0758
<i>Syzygium hancei</i> (韓氏蒲桃/紅鱗蒲桃), [Syh ⁺]	100	100	97.1	0.3363	1.5666
<i>Viburnum odoratissimum</i> (珊瑚樹), [Vo ⁺]	91.4	94.3	94.3	0.5399	1.5971
<i>Vitex quinata</i> (山牡荊), [Vq ⁺]	100	94.3	91.4	0.8140	1.6076
Total	96.99	84.86	71.14		

Upon completion of the construction work later on, this kind of damage to the seedlings was obviously seen reduced (Hau 2016). At the latest visit, the survival rates of 12 species remained higher than 70% as shown in Table 3. The relative height increment (RHI) was determined for each of the seedlings at the monitoring respectively at the 15th month and 33th month after plantation. The RHI was estimated using the equation proposed by Hoffman et al. 2002 as follows:

$$\text{RHI} = \ln (H_2 - H_1) / \text{year} \quad (1)$$

where H_1 and H_2 are the initial and final stem height in centimeters. The mean RHI for each species are given in Table 3.

4 TREE PITS

In the remaining area outside the trial site on the slope, there were tree pits provided filling with imported CDG without cement additive for growth of bigger trees on the slope. Tree pits measured 2m by 2m and approximately 1 m in depth at 8m spacing in centres, and staggered in arrays on the slope (Figure 3).



Figure 3: Tree pits of 2m x 2m x app. 1m deep at 8m spacing staggered in rows on the slope

5 DISCUSSION & CONCLUSION

A site trial was carried out to plant and monitor 20 native species of trees and shrubs on the subject soil cement fill slope. 11 species were shortlisted from the review of previous studies and works of tree planting on soil cement slopes. The shortlisted species were all considered suitable for growth on soil cement fill with a high survival rate. The other 9 species (highlighted in yellow in Table 2) were native species adopted in the study for the first time.

17 species in the trial had a resulting survival rate exceeding 50%, a threshold rate of selection for planting on the soil cement slope. 9 of them had a survival rate $\geq 80\%$; and a mean RHI at the 33rd month ranging from 1.1 to 1.6.

Over a monitoring period for 33 months, the growth rates (i.e. mean RHI33), of the 9 local species were all performing comparable to, or even slightly better than those of the other 11 shortlisted species from other studies or works. The mean RHI33 for the former ranged from 0.8998 (Dv) to 1.6076 (Vq) whereas for the other 11 shortlisted species from 0.3679 (Pa) to 1.543 (Cm). The survival rates and mean height increment in 33 months for the trial species are given in Tables 4 and 5 respectively.

Besides, for those trees that were growing in the tree pits, they were found growing equally well as if they were planted on a soil slope without any cement additive perhaps given the CDG fill in the pits.

The slope greening works to the soil cement fill slope in this project were found satisfactory. A photo of the slope after the greening works is shown in Figure 5.

It has been verified by the site trial that most of the 20 native species are suitable for growth on soil cement fill slope and 9 of them performed better than others, showing markedly high values of survival rate and mean RHI.

Scientific name (Chinese name), code	Date: 02.10.2015	Date: 20.12.2016	Date: 07.08.2018
<i>Vitex quinata</i> (山杜鵑), [Vq]			
<i>Cyclobalanopsis myrsinifolia</i> (小葉青岡), [Cm*]			
<i>Daphniphyllum calycinum</i> (牛耳楓), [De]			

Figure 4: The growth of trees and shrubs in the pilot study on the site trial



Figure 5: General view on the greening of soil cement slope
(photo taken in Aug 2018, about 3 years after plantation)

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A Review of Advances in Shallow Seismic Investigation for Managing Ground Risk

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ABSTRACT

Complementing traditional approaches to site characterisation, recent developments in near-surface seismic investigation techniques have led to better representation of ground structure and in-situ geotechnical properties, improving the likelihood of avoiding unforeseen ground conditions for infrastructure development. The civil engineering sector has sometimes had a somewhat jaundiced view of geophysics, but a properly designed, executed and communicated (and critically, early) geophysical screening strategy can help not only to optimise subsequent intrusive campaigns but to populate what can represent large information gaps (uncertainty) in the Ground Model space between points of control such as boreholes and probings and their associated localised tests. For existing and future infrastructure developments, looking ahead, the evolution of legislation relating to seismic site assessment and site vulnerability for dense urban areas such as Hong Kong necessitates an awareness of the art of the possible for geophysical investigation. We review six seismic methods where recent advances in data acquisition technology and/or processing offers an opportunity for improved early screening for infrastructure development and / or seismic site assessment, and benefits in terms of better risk evaluation and timely decision-making.

1 INTRODUCTION

Globally, infrastructure projects overrun on initial cost estimates by an average figure of about 30% (Flyvbjerg, 2004). Whilst the ground is but one contributory factor, significant cost overrun at the design and construction phases can be attributed to unforeseen problems associated with the ground. Geophysics allows wider spacing of intrusive investigations to be adopted where ground conditions appear to be relatively uniform, and justifies more closely spaced, targeted intrusive investigations where they are indicated to be complex and critical (Eddies and Wood, 2014). When adopted as a complementary technique to borehole drilling, the resulting reduced number of boreholes, samples and laboratory testing often means that a more reliable ground model can be developed for a similar or indeed lower cost to a conventional borehole programme. Following site screening using geophysics, an intrusive programme can then be targeted to effectively characterise the site at the right cost. In many circumstances, this not only means that the number of boreholes is reduced (with commensurate benefits to schedule, cost and risk), or at least better distributed with the value of each borehole being increased by being located better, but also leads to an intrinsically better Ground Model – with the potential for example in Hong Kong, to reduce uncertainty in the stratigraphy and properties variability associated with a buried, non-engineered reclamation interface. In turn, detailed information from boreholes can then be used to calibrate the geophysical information and allows constrained interpolation between points of control, further improving the Ground Model (Eddies and Wood 2014) and enabling earlier and better progressive design decisions to be made.

Increasingly, owners and developers are seeing the benefits of reducing ground uncertainty

and avoiding unexpected design and construction phase costs by the early application of screening geophysics, particularly seismic techniques, some of which have been developed from the oil and gas and nuclear sectors and are now being frequently applied to help build better digital representations of ground risk for infrastructure development.

We review and present six key advances in non-intrusive shallow seismic investigation including their applications in geotechnical problems; surface wave tomography (SAVM), interferometric surface wave analysis (IMASW), multicomponent (3C) broadband seismic, wireless (cable free) seismic, ambient vibration array measurement and H/V microzonation.

2 3D SURFACE WAVE TOMOGRAPHY

Surface waves (Rayleigh and Love waves) have been measured to determine the shear-wave velocity of the ground since the mid-1980's (Heisey et al., 1982), the data used to illuminate areas of less stiff, unconsolidated near-surface materials. Currently the multi-channel analysis of surface waves (MASW) technique is commonly employed in ground investigation projects (Park et al. 1999). This method involves the recording and processing of Rayleigh waves, the frequency/velocity characteristics of which are used to derive shear wave velocity through numerical inversion based on the Rayleigh wave equation that describes the relationship between Rayleigh wave and shear wave velocity. The MASW method, for a single receiver spread, generates a 1D shear-wave velocity profile of the ground. By moving the geophones and source over a target area, a series of 1D profiles can be collected and then combined to form a 2D profile image of shear-wave velocity and subsequently shear modulus using an assumed or measured bulk density.

The recently developed surface wave tomographic imaging method (Rector et al. 2015) allows for the targeted mapping of structurally weak ground on a 3D rather than a 2D profile basis. The tomographic approach, also known as the square-array void mapping technique (SAVM), uses geophones arranged in a square or rectangle around the target area with source position progressing around the outside of the rectangle (Figure 1, Figure 2).

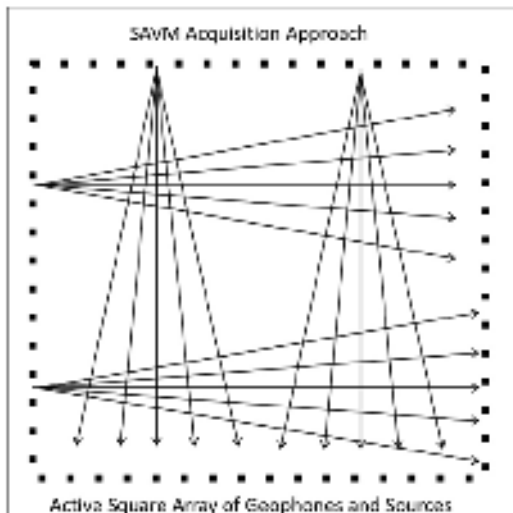


Figure 1: SAVM uses a square or rectangular array of seismic sources and receivers to investigate a 3D block of ground below the surface (Rector et al. 2015)

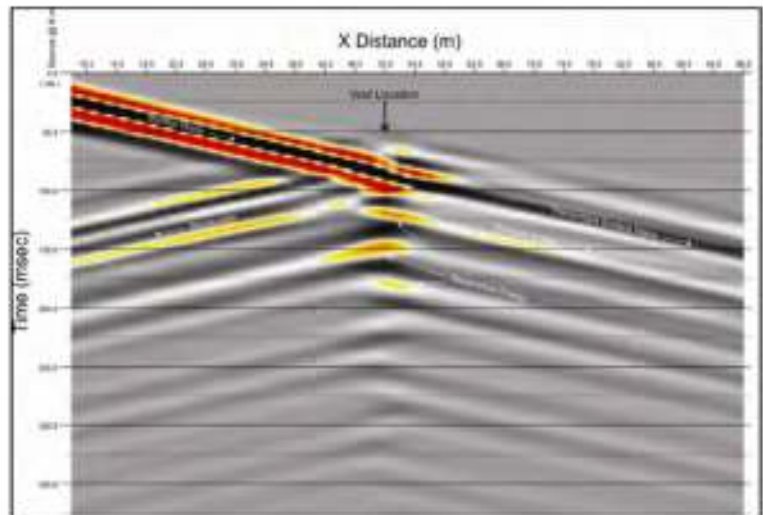


Figure 2: Numerical modelling to show that voiding leads to decreased amplitude of transmitted surface wave energy, high amplitude resonance and presence of backscatter (Rector et al. 2015)

This new geometry uses tomographic imaging of surface waves to reconstruct surface-wave velocity and attenuation profiles in three dimensions. In this way seismic wave propagation can be investigated throughout the target area in 3D by processing the data using a tomographic method, the median seismic wave frequency indicating the ground penetration depth. Previously, if a 3D investigation were required, a series of 2D MASW profiles would have to be collected in parallel to each other and then the results combined to a 3D model, this is time-consuming compared to the SAVM approach, but also

features between the parallel seismic lines could be missed and not included in ground models and subsequent decisions on ground risk.

SAVM has been used successfully within the mining sector to identify abandoned mine tunnels beneath open-pit benches (Rector et al. 2015). Numerical modelling showed that voiding could lead to anomalous amplitude and frequency response (Figure 3). In this case study, known voids proven by tightly spaced infill drilling were successfully identified by the SAVM method. Voiding was identified by the presence of attenuated signal, lower velocity and resonant energy:

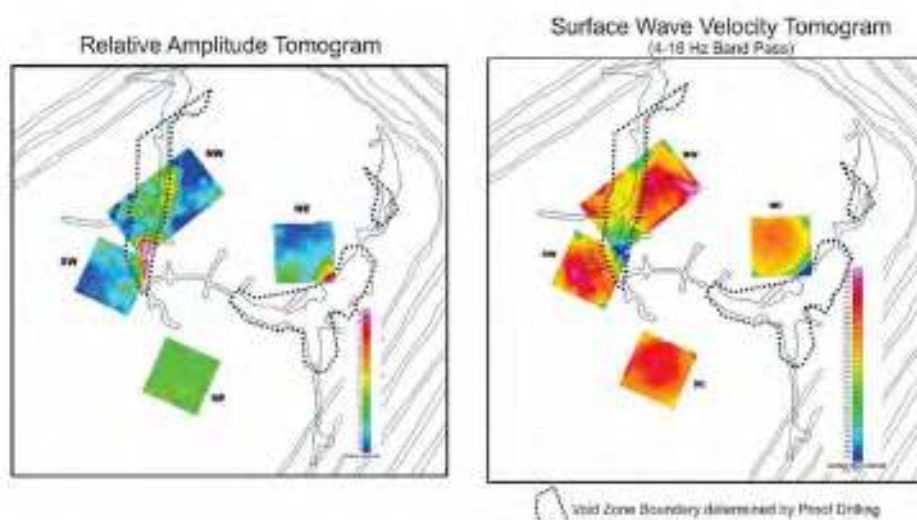


Figure 3: SAVM response over mined void characterised by anomalous amplitude velocity characteristics (Rector et al. 2015)

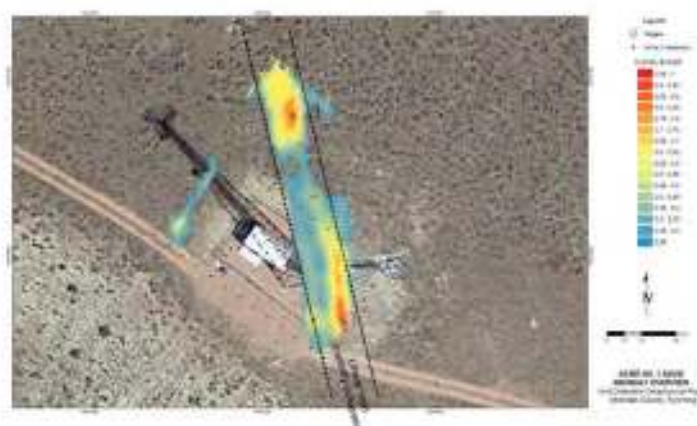


Figure 4: SAVM response over mined void characterised by anomalous amplitude and velocity characteristics (Source: Fugro)

One of the key benefits of SAVM is its ability to target specific surface areas to investigate, for example, building footprints or pile groups. Additionally, a matrix of SAVM squares or rectangles can be used to characterise a broader site, used in conjunction with complementary methods such as microgravity and or resistivity. Future work to develop SAVM will likely focus on enhancing the method by using the scattered wavefield to improve imaging resolution and integrating 3D surface wave analysis with other wavefield elements such as refracted waves (fig. 5).

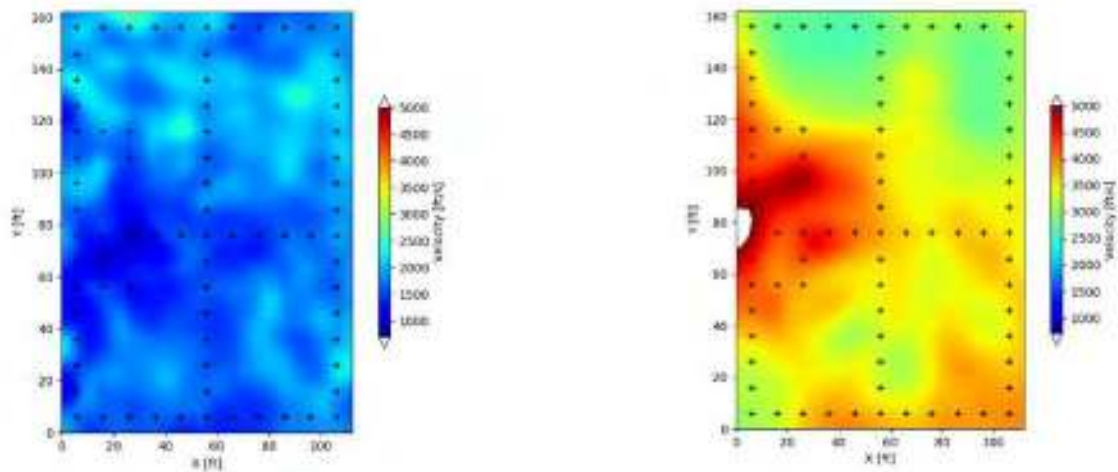


Figure 5: 3D P-wave refraction tomography imaging developed from the SAVM method showing relatively slow velocity characteristics 3 m depth (left panel) and a significant velocity high due to bedrock topography at 15 m (Source: Fugro)

3 INTERFEROMETRIC SURFACE WAVE ANALYSIS (IMASW)

Ground motion prediction equations commonly require estimates of average shear-wave velocity (V_s) to estimate ground-shaking parameters (Abrahamson et al., 2008). Averaged V_s to 30-m depth (V_{s30}) is a standard parameter used in ground-shaking hazards for Next Generation Attenuation equations (NGA, 2008) and the International Building Code (International Code Council, 2006). Site specific ground motion characterizations are therefore dependent upon efficient and robust methods to estimate site-specific V_s depth profiles to at least 30-m. Multichannel analysis of surface waves (MASW; Park et al., 1999) and refraction microtremor (ReMi; Louie, 2001) are standard-practice non-invasive methods for measuring shallow shear wave velocity.

In-situ small strain stiffness is a key parameter for geotechnical design. Implementation of surface wave investigation to investigate small strain stiffness is now commonplace and largely carried out through an MASW approach as previously described using active sources or refraction microtremor (ReMi) using ambient sources. Whilst MASW has been shown to be a robust method in many varying geological scenarios, there are some not uncommon circumstances where depth penetration is limited to 10-15 m. In addition, ReMi investigation is highly reliant on the ambient wavefield characteristics that might not always suit site-specific technical objectives.

Determining shear wave velocity characteristics to a depth of 30 m (V_{s30}) is also a key parameter for characterizing the effects of sediment stiffness on ground motions for seismic hazard analysis and is built into various international design codes such as Eurocode 8. Stiffness is an integral component of many seismic analyses, including site classification, hazard analysis, site response analysis, and soil-structure interaction (Wair & Schantz, 2012). The V_{s30} value can be derived using borehole geophysics methods including wireline velocity logging, P-S suspension logging, cross-hole seismic and down hole seismic, providing only the V_s profile for that immediate locality. So seismic profiling using surface waves, notwithstanding depth investigation limitations, is a very attractive technique where lateral variability in geological conditions might be expected such as those found in Hong Kong with variable tropical weathering.

To overcome the depth limitations of MASW and similar variants, a newly developed technique, interferometric multichannel analysis of surface waves or IMASW (O'Connell & Turner, 2011) was developed by Fugro combining the strengths of MASW and ReMi and introducing a new processing workflow. Surface wave data are collected in a similar way to produce 1D shear wave velocity profiles that can be combined to 2D profiles but yield significantly greater depth penetration with better resolution. This is achieved by combining the data collection and processing routines of a conventional MASW survey that uses active seismic sources, with those of an ambient noise survey (or induced noise in the absence of ambient noise). Compared to MASW acquisition, a great number of records are collected at larger offsets from the first and final geophones and recording time is significantly increased. The

IMASW approach (largely time domain cf MASW's time domain basis) improves resolution of low-frequency Rayleigh-wave phase slownesses (or better velocity estimates) by obtaining deterministic correlation Green's function (CGF) seismograms from passive and active-source records (O'Connell & Turner, 2011). MASW largely works with phase velocities; IMASW allows robust estimates of group velocity to be made, from which phase velocity can be determined as a derivative.

A Fugro study of IMASW data compared with P-S suspension logging data at a site in across the Van Norman Complex in northern San Fernando Valley (Figure 6) showed that, on average, 30 m depth shear-wave velocity estimates between the two methods differed by <1% (O'Connell & Turner, 2011). IMASW processing and velocity inversion approach is likely to provide robust estimates of 3–5-m depth averaged Vs structure to at least 100 m (Figure 6).

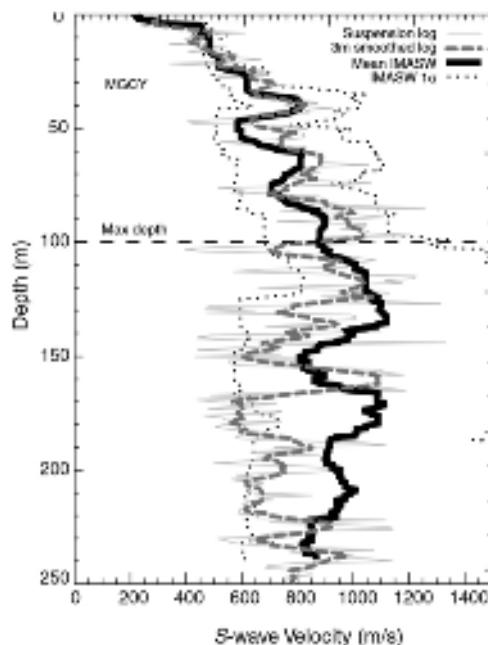


Figure 6: IMASW estimates of small strain Vs compared with PS Logging estimates of Vs – note strong similarities above 100 m depth ((O'Connell & Turner, 2011)

For geotechnical characterisation or seismic hazard studies, the IMASW method provides a robust non-intrusive approach that through the advantages of interferometry yields realistic Vs-depth estimates comparable to 2-3 m depth averaged Vs estimates derived from borehole data.

4 MULTICOMPONENT BROADBAND SEISMIC

Several technologies developed for the oil and gas sector are now emerging in engineering geophysics for near surface applications including geotechnical and hydrogeological characterisation. Traditionally, shallow seismic surveys would involve just a single component of the available wavefield induced by a seismic source of some type. Such surveys might yield an image of reflectors and discontinuities (seismic reflection imagery), a profile of layer velocities (shear or compressional wave refraction imagery) or a depth profile of shear wave velocity determined from Rayleigh wave behaviour (MASW and variants).

The development of multicomponent broadband seismic (e.g. Eddies et al 2018) for the near surface allows simultaneous collection of all wavefield components necessary for reflection, refraction and surface wave analysis. The new approach allows combined stratigraphic and structural imaging and screening of geotechnical properties in a single-pass, facilitating integrated geophysical models of structure and properties and higher confidence in derived geotechnical parameters.

The development of MEMS (micro electrical mechanical system) receiver technology (Figure 7) for hydrocarbon exploration means that in addition to full capture of the seismic wavefield in a spatial sense (along three orthogonal axes), the wavefield can also be fully captured in a spectral sense.



Figure 7: MEMS receiver technology allows simultaneous capture of seismic information along three axes and from very low to very high frequencies and opens up the opportunity for simultaneous imaging of structure and geotechnical conditions (Source: Sercel).

MEMS technology offers advantages over standard geophone-based receivers in terms of a flat frequency response, a very broad bandwidth and less sensitivity to off-vertical tilt. Extending from close to zero up to about 800 Hz. This brings an important advantage for near surface investigation in that high frequencies are desirable for imaging of layering and discontinuities whilst low frequencies are desirable for the screening of geotechnical properties. Using a MEMS-based approach allows all spatial components and all spectral components to be acquired simultaneously.

The multicomponent broadband approach using multicomponent MEMS technology was applied to a highly sensitive project within Europe (Figure 8) with the twin objectives of a) confirming a lack of

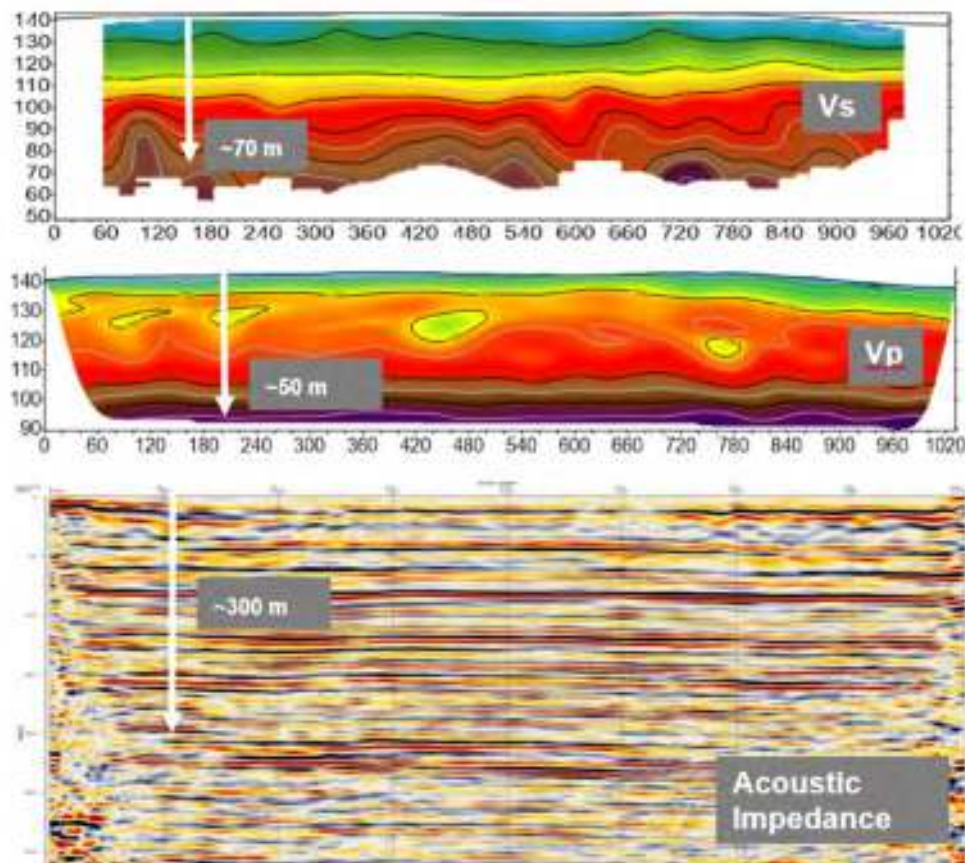


Figure 8: Multicomponent seismic profiling results from a single pass includes Vs distribution to about 70 m (top), Vp distribution to about 50 m (middle) and seismic reflection imagery to more than 300 m (bottom). (Fugro, 2017). Vs and Vp can be combined to produce distributions of small strain Poisson's ratio. Using a known or assumed density, similar distributions can be derived inter alia for shear and Young's modulus. (Source: Fugro)

faulting / continuity of stratigraphy in the top 300 m and b) geotechnical screening to at least 50 m. Time savings compared to individual surveys amounted to about 30% with better depth penetration for the surface wave / stiffness component.

5 WIRELESS (CABLE-FREE) SEISMIC

Large scale onshore seismic surveys have long been used for oil and gas exploration and larger civil projects to determine rock and soil structure in the near surface. Conventionally these surveys use geophones connected by cables to a recording system, which in a highly urbanised area such as Hong Kong would disrupt business and local communities. This is something construction firms are keen to avoid. In addition, damage to expensive seismic cables and equipment, which may have to cross roads, rivers and heavily trafficked areas, is a real project risk with consequences for both project cost and schedule.

Multiple wireless systems have recently been developed to avoid these problems, the earliest in use within the oil and gas industry since the early 2000's (Heath, 2008), and with time have decreased in cost, become more reliable and far more battery efficient. These systems typically divide in to two categories, those which communicate and send live data back to a central unit for data quality control, and those that require to shoot 'blind' and all data is collected at the end of the survey when geophones are retrieved (Crice, 2014).

Sirius Minerals' York Potash project (Figure 9) highlighted the use of 'light footprint' systems to help manage complex stakeholder environments whilst not compromising on the technical deliverables – an *unobtrusive* approach applicable to both environmentally sensitive and urban locations alike.

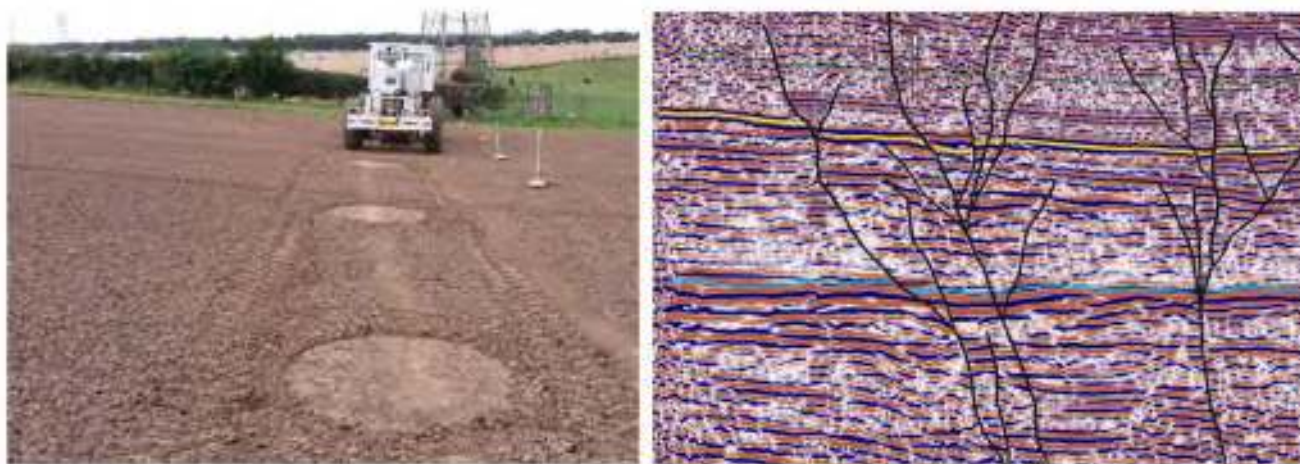


Figure 9: Data acquisition for one of Europe's largest mining projects. Wireless seismic receiver systems coupled with modern, lightweight seismic sources provide an unobtrusive 'light footprint' approach to seismic acquisition for identification of near-surface faulting and stratigraphy that can be applied to rural and highly urbanised environments alike. Note the absence of cables and individual receiver units connected to the recording system using radio and WiFi communication (Source: Fugro).

For the York Potash development, real time data analysis/QC allowed for careful choosing of seismic shot timings to avoid cultural and wind noise and monitor conditions of geophones which may be subject to interference by wildlife. The flexible nature of this system allowed seismic lines to cross roads and topographically challenging terrain as well as traverse heavily trafficked / built-up areas without disruptive and expensive traffic management and barriers to protect seismic cables.

Finally, wireless systems require the deployment of less ground equipment than for the equivalent survey with a cabled system this allows site teams to be smaller and subsequently cheaper while at the same time increasing production due to fewer equipment (cable) issues and fewer access challenges.

6 AMBIENT VIBRATION ARRAY MEASUREMENT (AMV)

V_{s30} is a key parameter to perform seismic site classification and requires measuring shear wave velocity from surface to at least 30 meters. Traditional techniques such as crosshole and active seismic can be expensive and not always easy to apply. Since the 1960's, techniques based on the analysis of microtremors recorded by arrays of very sensitive seismometers have been developed to obtain V_s profiles to a depth only limited by array size. Frequency Wavenumber (FK; Lacoss & al., 1969; Horike, 1985), High-Resolution Frequency Wavenumber (HRFK; Capon, 1969) and Spatial Autocorrelation techniques (SPAC; Aki, 1957; Bettig & al., 2001) uses the dispersive characteristics of the surface waves crossing the array to retrieve dispersion curve (phase velocity as a function of frequency) that is next inverted into V_s profile. Processing techniques and software have been improved over the years to reach a good level of maturity. Strong reliability of the results has been highlighted by successive research projects (SESAME, 2004; NERIES, 2008; InterPACIFIC, 2018) and is now well recognized by the scientific community.

The case study presented below (Figure 10) shows AMV array deployed at a chemical industrial site. Measurement was performed using eight no. 3-dimensional seismometers installed successively following 4 concentric arrays configurations (R1 to R4 configurations) in order to investigate the targeted depth. Large arrays allow the characterization of big wavelengths corresponding to 2 to 3 times the penetration depth. Geometry of each configuration determines array resolution domain delimited by minimum and maximum wavenumber. Several configurations are set to provide several dispersion curves inside arrays resolution domains and finally in a large frequency range when put together. The concatenation of partial dispersion curves allows both key objectives to be achieved, reaching a depth greater than 30 meters and a good velocity resolution in shallow layers.

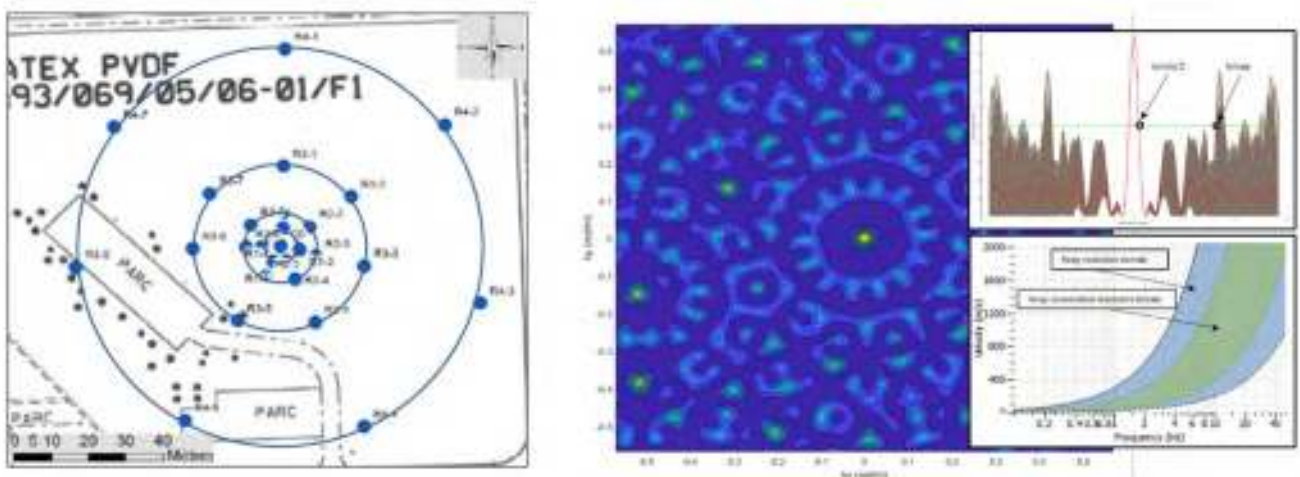


Figure 10: Typical passive receiver array geometry (left) with array configuration transfer function and resolution domains express as velocity / frequency. (Source: Fugro)

In this case, FK, HRFK and SPAC techniques applied to seismic data collected in the four array configurations provided dispersion curves between 4 and 30 Hz (Figure 11). These curves are inverted using a layered a-priori soil model (V_s , Poisson ratio, density) with a large degree of liberty accorded to model parameters. Inversion is performed using a neighbourhood algorithm (Sambridge, 1999; Wathelet, 2008). Additional data such as H/V ellipticity can be added to perform joined inversion. A misfit value is associated to every generated model and is used to select all models representative of the non-uniqueness solution of the inverse problem; typically all models with a misfit smaller than 1 can be considered as a good solution. AMV array technique finally provides a population of ground models (Figure 12) on which statistical analysis is conducted to retrieve a robust V_{s30} estimate with assessment of estimate uncertainty.

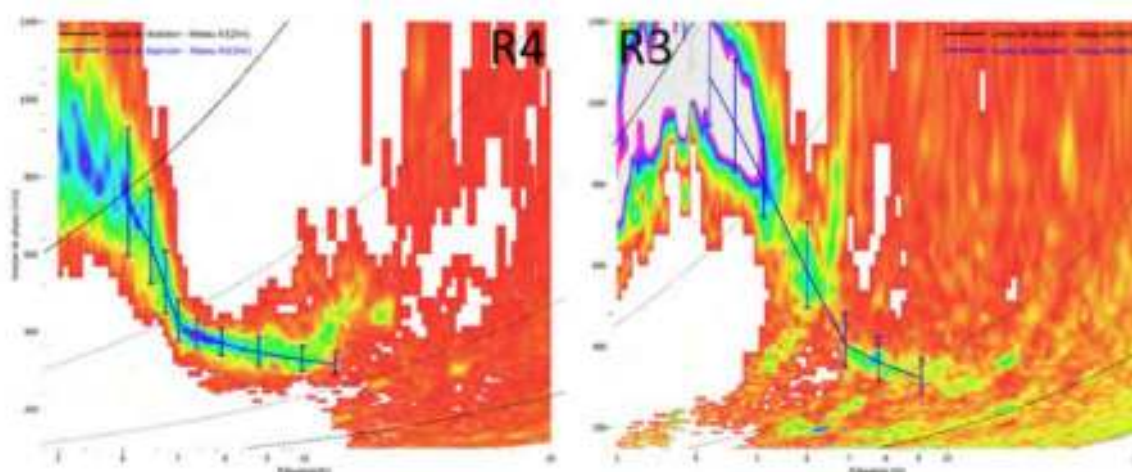


Figure 11: Dispersion curves obtain for arrays comprising 3 and 4 radii respectively, using the HRFK technique. (Source: Fugro)

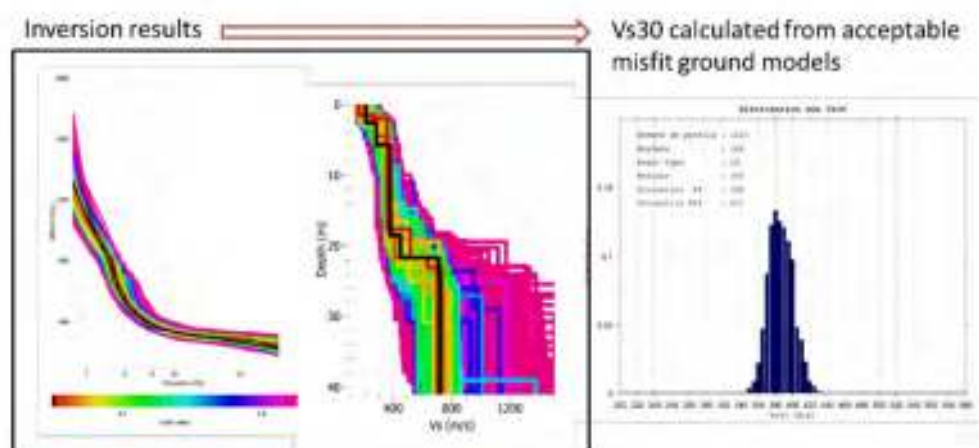


Figure 12: Inversion results and V_{s30} calculation. (Source: Fugro)

AMV array measurement bring major advantage in seismic site characterization because it allows measuring V_{s30} and V_s profile with no depth limitation and without a seismic source. Moreover, the method is non-invasive, can be deployed with no impact on the environment and is highly suited to highly urbanized environments such as Hong Kong. Numerical inversion remains the most challenging step of the processing workflow and should be performed by an experienced operator. To avoid this difficulty, some publications (Bard & al., 2010; Foti & al., 2018) suggest skipping inversion step and evaluate V_{s30} directly as the phase velocity corresponding to a wavelength of 40 or 45 meters ($V_{\lambda 40}$, $V_{\lambda 45}$).

7 PASSIVE SEISMIC – H/V MICROZONATION

One of the key objectives in seismic site characterization is to assess spatial variations of soil conditions over the site (Figure 13). To this end, the geophysical method of horizontal to vertical spectral ratio (H/V microzonation method) has been evolving since the 1980s (Nakamura, 1989). This method allows measurement of the fundamental frequency (f_0) of surface soil layers covering the bedrock provided the velocity contrast at the bedrock interface is sufficiently high. In these soil conditions, H/V spectral ratio curve exhibits a clear peak at the resonance frequency f_0 which can be mapped at site scale (spectral microzonation) performing measurement following a defined grid. H/V curves are also interpreted in terms of typology defined by curves character: flat curves generally correspond to outcrop bedrock, whereas multiple-peaked curves usually correspond to several strong velocity contrasts. Very sharp spectral peaks often relate to industrial noise (SESAME, 2004). Applying H/V methods provides a direct result in terms of soil conditions variability by analyzing curve typology and/or f_0 values distribution

(Figure 13). H/V measurement is typically about 20 minutes per surface station/data point and performed with high sensitivity seismometers.



Figure 13: H/V microzonation is highly suited to industrial or urban environments. In this example f_0 frequencies were acquired over a major industrial site. (Source: Fugro).

As part of H/V microzonation, f_0 frequencies are directly dependent on the bedrock depth and mean V_s velocity in the upper layers following the relationship described below (for a tabular, homogeneous, isotropic layer surrounding a half-space):

$$f_0 = \frac{V_s}{4H}$$

Where H = bedrock depth; V_s = mean V_s velocity in the layers overlying the bedrock.

If V_s is known (by crosshole or passive seismic array for example), then the measure of f_0 allows the estimation of bedrock depth at each measurement point. H/V measurements can also be directly calibrated with boreholes by defining a relation between bedrock depth (provided by boreholes) and f_0 frequencies (Figure 14). H/V microzonation results, integrated with other ground data provides an opportunity to create a map of bedrock elevation over a site assuming a significant impedance contrast at the bedrock top (Figure 15). The H/V method is suitable for application in any environment but is particularly suited to noisy and urbanised areas such as Hong Kong.

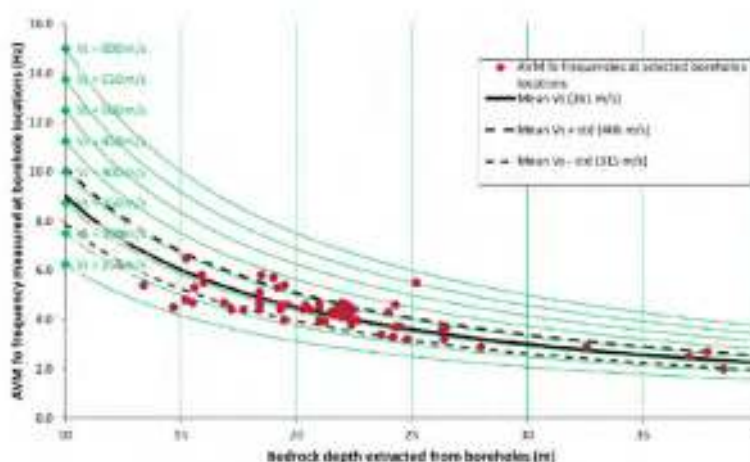


Figure 14: f_0 frequencies measured at boreholes location and equivalent f_0 calculated for different V_s values in sand cover compared to bedrock depth extracted from boreholes (Source: Fugro)

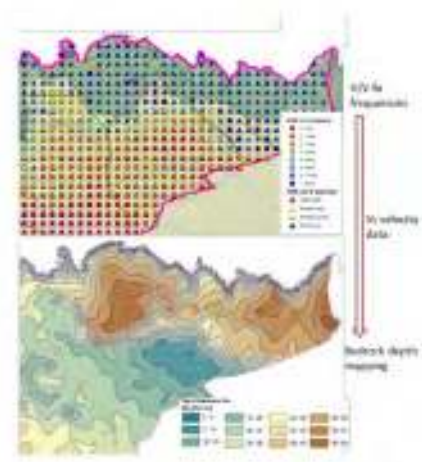


Figure 15: Bedrock elevation mapping using f_0 frequencies measured by H/V microzonation and mean V_s velocity determined from H/V microzonation and existing geotechnical data (Source: Fugro)

8 CONCLUSIONS

Early phase geophysical screening including seismic techniques provides a complementary approach to direct geotechnical investigation. This review has summarised the state of the art for applying seismic techniques to solving near-surface problems of structure and conditions, whether in an infrastructure development context or as part of seismic vulnerability assessment. Geophysical screening has been successfully applied across a number of environments including highly urbanised locations and complex subsurface geology such as Hong Kong. Building a more robust ground model with inter alia seismic techniques will not only help projects improve on a (globally poor) record of cost overrun by helping to avoid the unforeseen, but will bring about a number benefits for all major project stakeholders including owners (improved risk transfer), consultants (earlier, progressive design) and contractors (competitive pricing of ground risk).

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Quantification of the Risk in the Marble Sites by Geostatistical Analysis

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ABSTRACT

Karst morphology is a known geotechnical problem in Hong Kong, a design and checking system has been adopted in practice using Marble Quality Data (MQD), which is a combined measure of the dissolution voids effect and mechanical implication of fractures. A spatial variography analysis has been conducted on the MQD data derived from a marble site in Yuen Long area, it is found that the spatial variation of MQD dataset is horizontal anisotropic and can be modeled by a combination of a nugget effect model and exponential models. It is proved that the variogram models can be used to describe the spatial variation characteristics of MQD data.

The obtained variograms has been used to estimate the MQD data at the un-sampled positions through geostatistical kriging and simulation. A 3D estimation geotechnical model can be constructed based on the derived spatial variation and used for marble site evaluation, and the capability of providing a potential tool to quantify the risk makes the estimation method favorable.

1 INTRODUCTION

Karst morphology is a known geotechnical problem in Hong Kong. The common engineering works in these areas involves ground investigations, excavations, foundations and groundwater pumping, and these engineering works may become difficulty tasks due to the existence of marble and karst features, the reported main geotechnical issues include uncertain or discontinuous support, possibility of gross failure, buckling, deflection and tip damage of driven piles, and subsidence of ground surface (GEO 2007). However, construction in karst/marble designated areas is unavoidable due to the limited land supply. Scheduled areas have been defined to stipulate the construction works involving karst issues (BD APP-61 2009). On the past foundation works, bore piles have been sunk more than 100 m deep to pass through the karst cavity and reach the supporting layer, even though issues such as excessive settlements or sink hole caused by cavity collapse often cause delay or abortion of works (GEO 2006). The construction works in marble designated area are normally companied with high risk and high stakes, the success of the works requires the marble condition variation to be identified and incorporated in the early decision making process. To characterize the marble site and evaluate the risk for construction works, a site rating system has been adopted by local authority based on the Marble Quality Designation (MQD) value, which is derived from borehole investigation data (Chan 1994). The increase of the vertical effective stress at marble surface for foundations bearing on soil is also limited based on the marble classification category (GEO 2005).

The MQD classification system has been approved an efficient tool for general site condition evaluation through the past practice. The engineering work can be designed and checked with greater confidence through the MQD and marble mass classification system (Wong et al. 2011). In order to exploit the full advantage of MQD data, there is a need to understand the special characteristics and the

geotechnical logic residing in the MQD data. Meanwhile, In Hong Kong context, boreholes on a grid of 7m to 10m are commonly required to intercept the specific karst features for high-rise development on karst site (BD APP-61 2009). A comprehensive collection of geotechnical data from ground investigations throughout the territory of Hong Kong is housed by government agency. For instance, the Geotechnical Information Infrastructure (GInfo) maintained by GEO is an online GIS platform providing easy access of GEO geotechnical data (GEO 2018). The availability of the historical geotechnical data, although most of them are scanned documents, provides an opportunity for the marble data study. The concepts of MQD data and geostatistical variogram will be first described in the next two sections, then a case study will be presented, and the conclusions are summarized at the final part.

2 MQD DATA

The karst features normally include surface karst, overhangs, dissolution joints, deep cavity, underground channel and underground cavity. Based on the knowledge of karst process and common karst feature, the Marble Quality Designation (MQD) is defined using two main input parameters derived from borehole records, the Rock Quality Designation (RQD) and Marble Rock Recovery Ratio (MR) as following (Chan 1994):

$$\text{MQD} = \text{Average RQD} * \text{Marble Rock Recovery Ratio (MR)} \quad (1)$$

$$\text{Average RQD} = \sum_{L_1}^{L_2} r_i * \frac{l_i}{L_2 - L_1} \quad (2)$$

$$\text{MR} = \sum_{L_1}^{L_2} \frac{l_i}{L_2 - L_1} \quad (3)$$

Where l_i is the length of the core of RQD value

L_1 is the top elevation of the section of the core being considered.

L_2 is the bottom elevation of the section of the core being considered.

$(L_2 - L_1)$ is usually 5 m.

The MQD is a combined measure of the effects of the dissolution voids and the physical and mechanical implication of fractures on the marble rock, the rock masses can be consequently classified based on the MQD value using the table proposed by Chan (1994).

3 VARIOGRAM MODELING

Same as other geotechnical data, MQD data are associated with specific spatial locations in the ground, and spatial continuity exists in most earth science data sets. For a geotechnical attribute which exhibits continuous variation, the attribute values at two points close to each other are more likely to be similar than the attribute values from two points that are far apart. This spatial continuity can be quantitatively summarized through some indicators, in which the variogram is one such commonly used parameter (Isaaks et al. 1989).

The variogram measures the average dissimilarity between data separated by a separation vector. It is defined as the average squared difference between two attribute values and is expressed as a function of the separation vector as shown in Equation 3:

$$\gamma(h) = \frac{1}{2N(h)} \sum_{\alpha=1}^{N(h)} [z(u_\alpha) - z(u_\alpha + h)]^2 \quad (4)$$

where $N(h)$ is the number of pairs of data, u_α is the location, and $z(u_\alpha)$ is the data value at location u_α .

To obtain the variogram at any separation distance, a model needs to be inferred for computing a variogram value for any possible separation distance. Variogram model can be described through three parameters: sill, range and nugget effect. The modeling result can be expressed as the combination of

the basic models, the most frequently used basic models include nugget effect model, spherical model, exponential model, gaussian model and power model. The variogram modelling can be a complex process, it should be decided according to available information and modelling objective, and the modeling decision need to be clearly documented (Issaks et al. 1989; Goovaerts 1997).

As the soil variability information is a key factor for various decision making process commonly encountered in geotechnical engineering, the application of variogram study on geotechnical parameters has been extensively reported by former researchers (Ali et al. 1989; Soulie et al. 1990; Honjo et al. 1991; Oliver et al. 1991; Chiassen et al. 1995; Moustafa et al. 1998; Li et al. 1999, Przewłócki 2000; Gringarten et al. 2001; Angus 2013). However, the data availability and complexity of the modelling process is still the major obstacle for this analysis tool in geotechnical engineering (Hammah et al. 2006).

4 SPATIAL ANALYSIS

4.1 Geology condition

The study site is around 210m in length and 80m in width and located in the eastern part of Yuen Long, HK SAR and reported by Chan & Pun (1995). The regional karstic cavities mainly occur in layered marble of Lower Carboniferous which can be divided into 2 parts, the lower part is the Long Ping Member with a depth varying from -20mPD to -150mPD, the upper part is the Ma Tin Member with a depth varying from -8.3 mPD to -90 mPD. The Ma Tin Member mainly comprised of white, massive, pure marble with some dark grey dolomitic marble are characterized by highly to intensely karstic development, the most karst cavities recorded in the boreholes vary in height from 1m to 2m, and the maximum recorded height of a cavity is 24.5m (Lai 2013).

4.2 Data selection

A total of 500 boreholes have been sunk in this site area. In this study, the locations of 150 boreholes are extracted directly from the available borehole log reports extracted from GInfo, the data from those boreholes are subsequently used as a verification dataset. For other boreholes, the locations are derived from the location map in Chan & Pun's report (1995), and data from those boreholes are used as sample data in this study. Fig. 1 presents the available borehole locations in this study and the minimal distance between boreholes is around 1.5 meter.

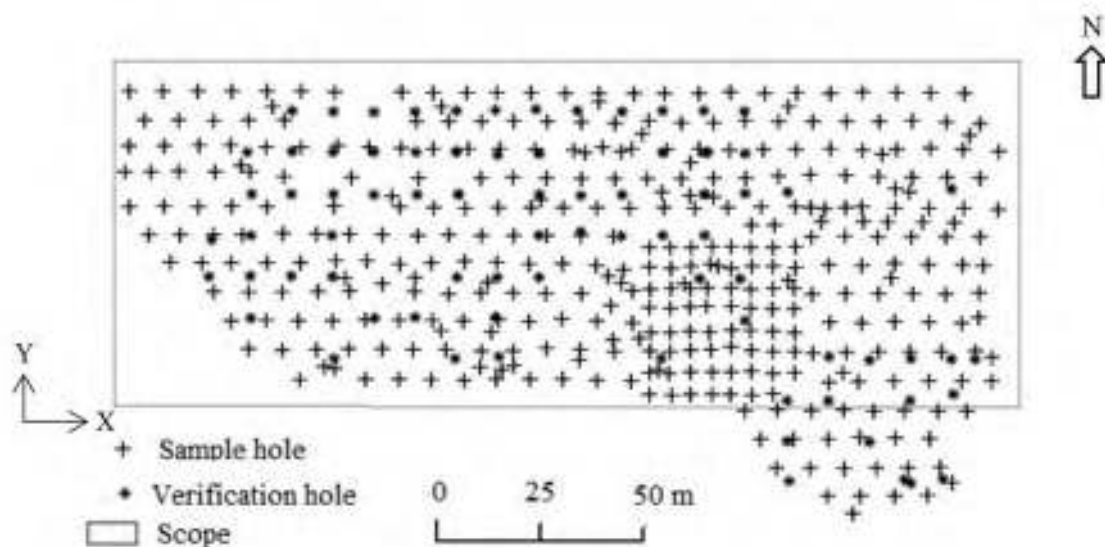


Figure 1: Borehole locations and study scope

Fig. 2 presents the scatterplots for the MQD sample data along the X, Y Z direction, the coordinate system is defined by the ENU (East, North and Up) convention. No obvious trend can be identified from scatterplot along the X direction, the scatterplot along the Y direction suggests that the data from south portion have a higher MQD value, the marble condition is much better around the south portion and the data has a different statistical characteristic with data from other boreholes. To ensure an effective rock head, 20m drilling below the Grade III marble is recommended in the HK practice, the preference sampling can be identified from the scatterplot of MQD data along the Z direction, the MQD data below -75.0mPD show good quality marble condition (ref. Table1) and less variation. The analysis need to be conducted on data from same population, that data from the south portion and data below -70mPD were hence excluded in this study. The scope of the study area is also presented in Figure 1 and there are 2224 MQD data from 352 boreholes for the sample dataset and 715 MQD data from 74 verification holes for the verification dataset.

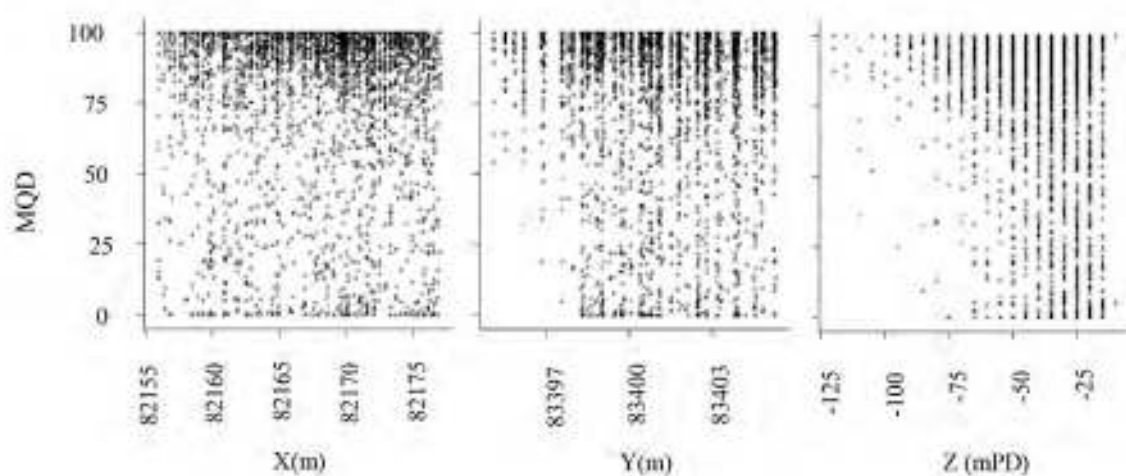


Figure 2: Scatter plot of the Sample Dataset

The mean value of the sample dataset is 75.3 and standard deviation is 31.6. The histogram of the sample data is presented in Fig. 3. The data is highly negative skewed with a skewness of -1.32, suggesting existence of lower MQD value of the dataset.

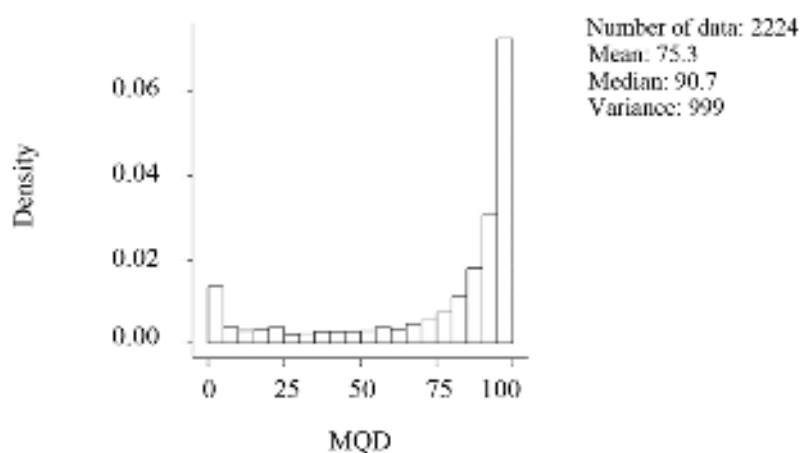


Figure 3: Histogram of sample dataset

4.3 Variogram

The variograms for a limited number of separation distances can be calculated from the sample dataset. Experimental variograms were calculated along various directions where the azimuth angle and dip angle are changed in every 30 degrees. Fig. 4 shows the calculated experiment variogram along the vertical direction, the variogram is standardized by the sample data variance. As the shown in Fig. 4, the experimental variogram reaches the sill at 20m and have a nugget around 0.45.

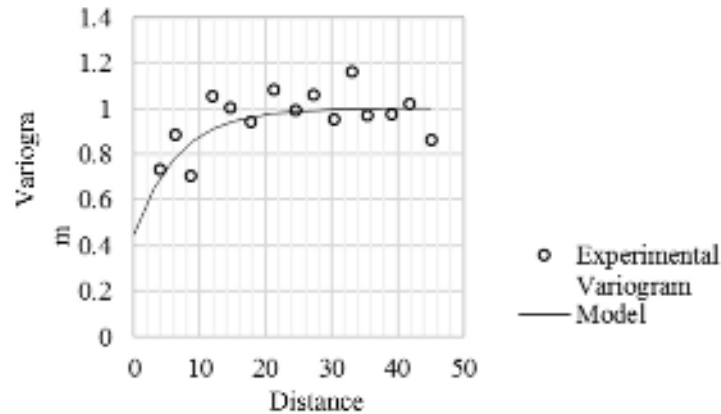


Figure 4: Vertical variogram

The experimental variograms in horizontal direction are presented in Fig. 5. The experimental variograms shows geometric anisotropy in the horizontal plan. Geometric anisotropy means that the range changes with direction, while the sill remains constant. The calculated results show 2 horizontal principle directions of maximum continuity: NE30 and NE120, and the minimum continuity can be found around N0E and N60E.

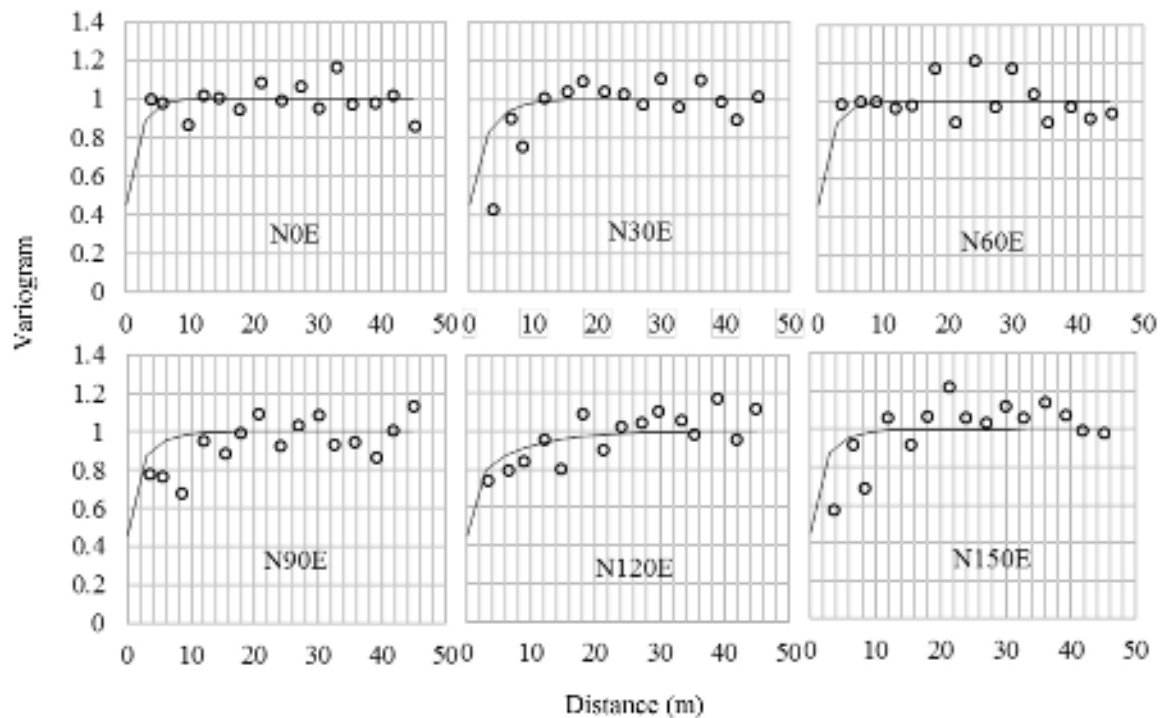


Figure 5: Directional horizontal variograms

Modeling anisotropy of the variogram requires the re-orientation of the coordinate axes (Issaks et al. 1989), the ZXY convention used in GSLIB (Deutsch et al.1998) was followed for the axes rotation in this study. A nugget effect model and 2 directional exponential models were used to fit the experimental variograms by trial and error, the parameters defining the variogram model are summarized in Table 2. The modelling results along different directions are also presented in Figure 4.

Table 1: Variogram model parameters

Model	Sill	Range X	Range Y	Range Z	Rotation X	Rotation Y	Rotation Z
Nugget	0.45	–	–	–	–	–	–
Model 1	0.275	23	5	20	120	0	90
Model 2	0.275	11	3	20	30	-90	-90

4.4 Estimation and Simulation

Variography study provides quantification tool for the spatial characteristics of the MQD data, the information is the key input for geostatistical estimation and simulation, by which the data spatial variation need to be honored. With no underlying spatial trend in the MQD data and assuming the mean to be unknown but constant within the neighborhood, an Ordinary Kriging (OK) has been conducted to estimate the MQD value based on the nearest data points from the sample dataset. The OK estimator estimates the expected field by minimizing the estimation error, the estimation results at the location of the verification dataset were extracted and presented in Fig. 6(a). With the high nugget effect of the variogram model, the estimated data are highly variable with a correlation coefficient of 0.60 with the verification data. As shown in Fig. 6(a), the lower values tend to be overestimated and the higher values tend to be underestimated, this is caused by the smooth effect of the linear interpolation estimator. Fig. 6(b) shows the error distribution of the estimated data with the verification data, the mean of the error is around zero and the error distributed symmetrically around the zero line, as suggested by the unbiased estimation characteristic of ordinary kriging.

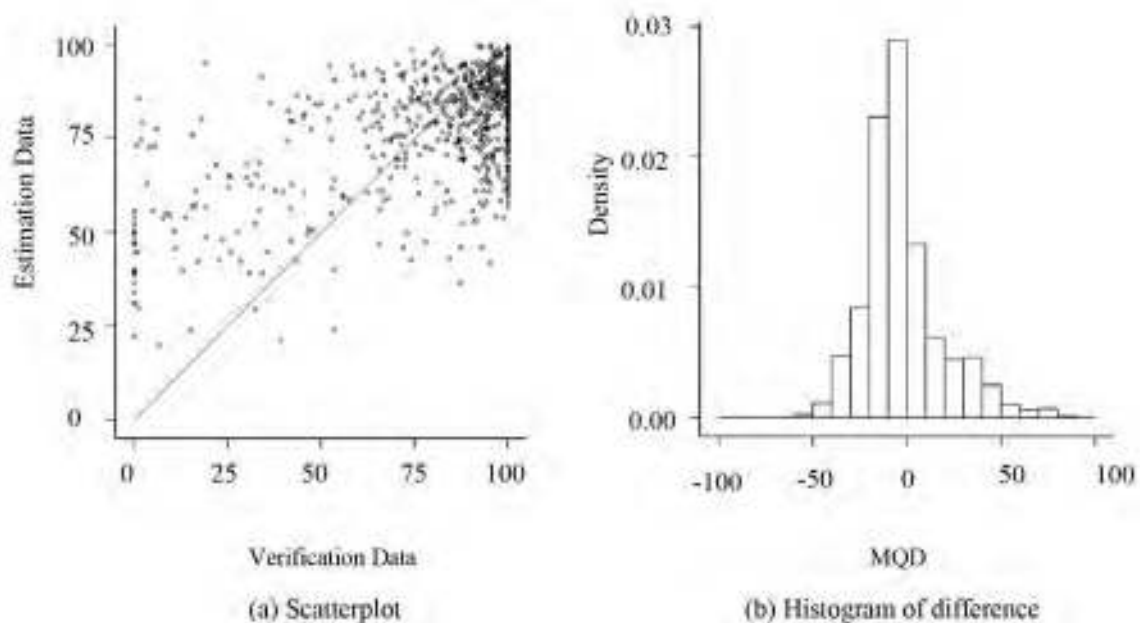


Figure 6: Ordinary kriging results

The uncertainty of the MQD data can be evaluated through developing simulated realizations of the random field. The popular Sequential Gaussian Simulation (Deutsch et al. 1998) was utilized for the MQD data evaluation. The sample dataset was transform by normal score transformation, the variogram analysis was conducted on the transformed data and applied in the simulation process. The simulation generates multiple equal possible realizations of a multivariate random function consistent with the sample data and the in situ variability. A cumulative distribution function (cdf) of simulated data for the interested location can be derived from the simulation results and consequently the uncertainty of the MQD data at the specified location can be evaluated. Fig. 7 shows the simulated results corresponding to the 3 percentile of the cumulative distribution function using the SGeMS (Remy et al. 2009). From Fig. 7 the location of areas with higher MQD values can be delineated, is should be a help for the foundation planning.

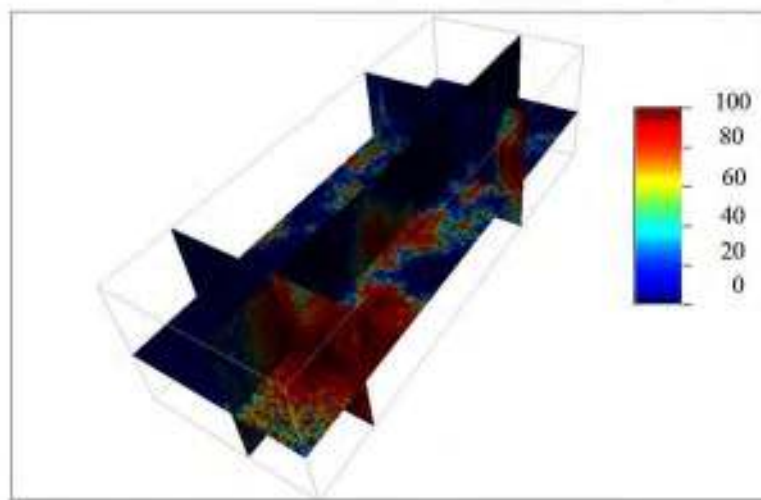


Figure 7: Simulation result (percentile =3)

Fig. 8 presents a comparison of the simulation data with the verification dataset, the result shows that most of the simulation results are below the 45 degree diagonal line and in the conservative side. By increasing the percentile of the simulation, a less conservative result can be obtained.

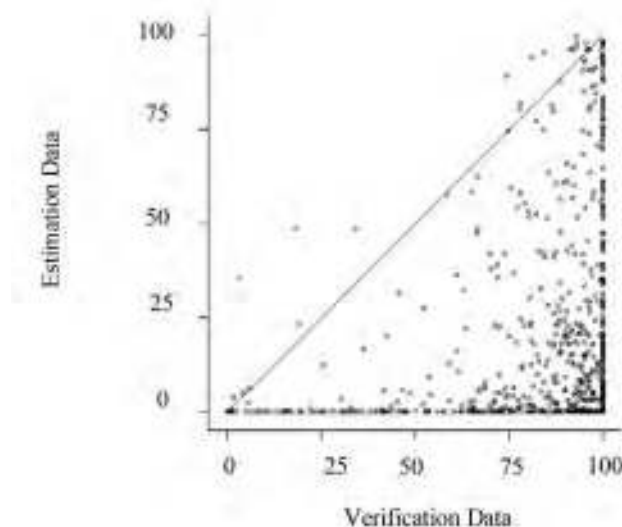


Figure 8: Scatterplot of the simulation result with the verification data (percentile =3)

5 CONCLUSIONS

A spatial characteristics study has been conducted on the MQD data derived from a Yuen Long marble site. The high variation of MQD data even for very close locations can be identified from the experimental varograms and represented by a nugget effect model. The geology heterogeneity is represented by directional geometry anisotropy and modeled by a combination of 2 exponential models. Although the data presents spatial characteristics, the high nugget effect suggests that data is highly variable. In practice, it means further reducing the borehole distance may not help improve the understanding of the site. By this case, 2-3m minimum distance would be enough to define the model.

The sequence Gaussian simulation demonstrates a quantity method for the uncertainty evaluation, it could be a potential help for a better understanding the risk for geotechnical works in marble conditions. By adjusting the percentile level, the designer actually can quantify the risk and make a balance between the cost and risk.

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On The Application Of UAVs For Rock Joint Mapping – A Case Study

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ABSTRACT

In the past decade, significant advances in Unmanned Aerial Vehicles (UAVs) have created many opportunities for a significant transformation in the application of this technology across geotechnical engineering practice. Unmanned aerial vehicle (also known as drones), are now widely available for both commercial and private applications. In this paper, the authors will describe how UAVs can be used to enable rock joints to be mapped digitally. The Digital Rock Mapping (DRM) process is described by way of a case study from the Anderson Road Quarry (ARQ). The process entails the creation and analysis of a detailed photogrammetry model. The paper critically analyses the results of a blind trial comparing the digital process against manual mapping of the same area. Conclusions are presented along with areas for further development.

1 INTRODUCTION

The first recorded development of UAVs was by the military as early as World War I. UAVs have since evolved beyond recognition from these early military applications to multi-disciplinary solutions in the public and civil domains. Examples include search and rescue operations, agricultural irrigation monitoring, construction monitoring and geoscience research. In recent years, the engineering geological community has started to use UAVs to develop photogrammetry models and document geological features in rock outcrops.

Photogrammetry is now a widely used method of data capture across the industry, but the methods are still largely undervalued. Representative digital models of sites are created for visualisation purposes, rather than creating an accurate model that can be used for detailed analysis with the accuracy and reliability of traditional survey grade measurements. Like anything digital, technology is improving at a significant pace and new methods are available all the time to improve both the accuracy and resolution that can now be obtained.

The close collaboration between survey teams and engineering geologists has never been so important. Traditional topographic surveys with a measurement every few metres are typically no longer undertaken for geological purposes. A UAV and resulting photogrammetry model now provides a point cloud with hundreds of thousands of measurements just a few millimetres apart. A traditional survey crew would be on site for months to gather the same data that a UAV gathers in just 30 minutes in the sky.

2 BACKGROUND

The use of UAVs in Photogrammetry and remote sensing techniques was first seen in the early 1900s and used until the late 1970s largely by the military (Colomina & Molina, 2014). The modern, commercial use of UAVs in photogrammetry and remote sensing was started about 30 years ago but the high cost of the data collection and constraints, in many cases, of remoteness and inaccessibility of sites limited its use. The early growth in the use of UAVs for photogrammetry and remote sensing was largely due to the advancement in telecommunication and computing technology Westoby et al (2012). It has however only been in the last 5 years or so that the increasing availability of low cost UAVs, high-resolution cameras, and off-the-shelf photogrammetry software means that reasonably accurate photogrammetry models and digital terrain models can now be created quickly and easily. The level of accuracy now achievable with photogrammetry in many cases negates the need for expensive LiDAR scanning.

In recent years, there have been numerous papers and reports on the increasing use of UAV and data processing using “Structure from Motion” (SfM) for 3D photogrammetry and remote sensing for landscape and geological mapping. The SfM permits a fast and inexpensive way to develop photogrammetry models from photographs. A photogrammetry model is a 3D point cloud where each point has its own x,y,z coordinates which together provide an accurate 3D representation of a site.

Research has been conducted into the accuracy of photogrammetry techniques in developing an accurate model with positive results for instances: Haneberg et al (2006) on a 3-D structural mapping for rock slope design along Interstate 90 in Washington, US; Tung et al (2018) to a rock slope in Malaysia; and Mezghani et al (2018) on a site investigation for oil and gas exploration in Central Saudi Arabia. Many commercial software packages are now available and used commonly in commercial applications.

Over the past few years there has been a variety of research projects undertaken to develop and improve the recognition of rock features from 3D point clouds. Tung (Tung et al, 2018) used FACETS, a structural geology plugin, implanted in the open-source software Cloud Compare, to extract planes from dense point clouds to map rock slopes.

The full power of the photogrammetry models are yet to be realised, however the industry is rapidly developing increasingly complex tools to access the data contained in the models. This paper discusses the development of a Digital Rock Mapping (DRM) tool to use the photogrammetry models to extract information on the joints within a rock mass.

3 SITE DESCRIPTION

3.1 Site Setting

Anderson Road Quarry (ARQ) is situated in the northeast of East Kowloon in between the Kwun Tong District to the south west and the Sai Kung District to the north east. The quarry was operational for over 60 years from the 1950s until its closure (Woods et al, 2018). The 90ha of land that the quarry covers is now being developed to provide approximately 12ha of usable land for residential building developments and associated facilities. The development works include the formation of about 40 ha of land platforms and associated geotechnical works; road works including construction of approximately 3-kilometre long vehicular roads, footpaths, cycle tracks, an approximately 130-metre long underpass at the southern end and a public transport terminus at the northern end of the ARQ site.

ARQ consists of a benched rock slope that was cut during different phases of the historic quarry excavations. The series of benches is aligned approximately northwest to southeast following the ridgeline along the northeastern side of the site. The benched slope is approximately 2km in total length and up to approximately 200m in height. The site setting of ARQ is shown in Figure 1.

3.2 Geological Setting

The published solid and superficial geology map (1:20,000) by the Hong Kong Geological Survey, indicates the site is predominantly granite of the Mount Butler Granite, laid down during the Early Cretaceous period. It is equigranular, with a fine to medium grained texture. The granite outcrops at the site formation level as well as the lower section of the quarry slope features in the north. The



Figure 1: Location of ARQ site and the Slope Feature No. 11NE-D/C998

granite approaches a sharp contact with volcanic rock at higher elevations of the quarry slope. The volcanic rock is described as coarse ash crystal tuff to eutaxitic fine ash vitric tuff which belongs to the Mount Davis Formation and was deposited in the Early Cretaceous. There are generally 3 major joint sets intercepting the quarry slope in both granitic and volcanic outcrops (Woods et al 2018). The rock joints in both the granite and volcanic materials have predominantly been found to be tight and clean to surface stained. The joints in the granite were typically rough and whilst those of the volcanic rock were more typically smooth and planar. Clay infills have been observed but these have been confined to limited and specifically orientated joint planes or associated with minor shear zones. Minor shear zones are occasionally observed subparallel to a major joint set.

3 PHOTOGRAMMETRY

As previously described, digital photogrammetry models contain a huge amount of data. When this comes to rock faces, the data contained within the model can be used to measure the orientation of the defects, joints and planes. The Slope Feature 11NE-D/C998 and its nearby areas within ARQ site provided the opportunity of a trial site to develop the new technique of extracting the data on the rock mass characteristics of the site directly from the photogrammetry model. This was undertaken by developing a variety of detailed algorithms discussed later.

A photogrammetry model is constructed from many individual photographs. Each photograph typically overlaps with the adjacent photographs by a minimum of 60%. The production of a detailed model starts with gathering the data. At the ARQ site we used a Phantom 4 Pro UAV to fly the site, which is a quadcopter UAV mounted with a 20 megapixels and focal length of 8.8mm camera able to take photographs to a reasonably high resolution from the heights for this study, however it is ultimately the skill of the UAV pilot that ensures the accuracy of the data. Photographs need to be taken from every possible angle around the rock face to ensure there are no gaps in the model. Pix4D is one of the many photogrammetry processing software packages available and it is the one that was used in this study.

The accuracy of photogrammetry models is achieved by incorporating surveyed ground control points. The number of ground control points to use for a particular site comes with experience but for sites with significant altitude changes such as cliffs, typically four to six points are sufficient for most engineering applications. This would increase for long linear sites such as infrastructure sites. From experience we have found that for most sites, there is a tipping point where the inclusion of additional ground control points adds no additional accuracy to the end model, but significantly increases the processing time to create the model.

The data for the photogrammetry model was gathered by a UAV contractor. As the model was being created for the purposes of trialing the Digital Rock Mapping (DRM) software, we did not need to model

the entire quarry and therefore an area of the quarry of approximately 1.5ha was chosen. The height difference between the top slope crest and the base of the quarry floor in this region was approximately 80-100m. The UAV gathered the required data in just 1.5hours, firstly along a flight path above the slope at 125m and 135m above the quarry floor to gather the data from the vertical angle. The UAV was then flown horizontally alongside the rock faces at a distance of approximately 20m at various elevations from the top of the slope down to the quarry floor. This was to gather the detailed information from as many other angles and orientations as possible.

The Pix4D software was then used to generate the photogrammetry model, from the photographs and ground control points, refer to Figure 2.

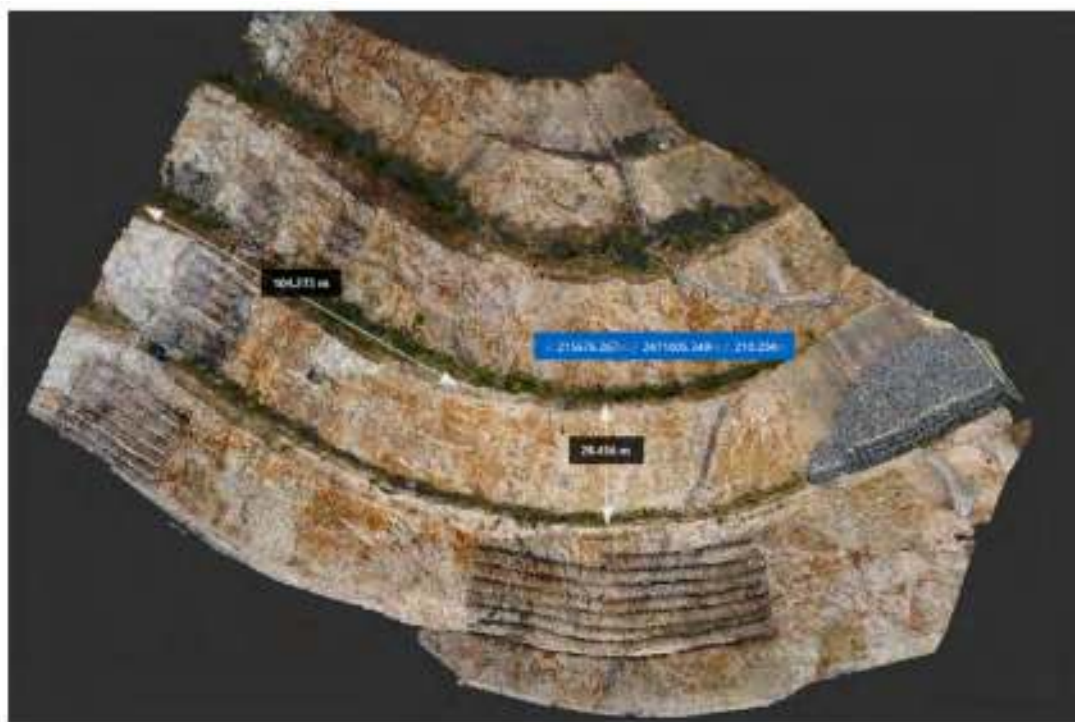


Figure 2: Photogrammetry model of a portion of the ARQ site

4 DIGITAL ROCK MAPPING TRIAL

A portion of the quarry face was selected measuring approximately 30m across and 20m high with an overall slope angle of 65° . The processed photogrammetry models were run through the DRM software which locates and measures the joints and planes. The processing takes varying amounts of time depending on the size of the model, however once completed the results are shown in a highly visual environment. The digitally located planes are individually coloured based on the orientation and it is therefore quick to visually identify the key joint sets. The orientation of each plane is measured and plotted on an on-screen, interactive stereonet which allows the user to select certain data points that are then highlighted on the 3D model, refer to Figure 3.

The results of the analyses are imported into widely used, commercial software such as rocscience Dips. The conventional analysis of the data is then typically undertaken.

5 VERIFICATION

For the purposes of this study, a blind trial was undertaken to compare the results of the DRM against manual mapping. As with any software, the results should be verified and the same section of the cliff face (the area marked yellow within Slope Feature No. 11NE-D/C998 in Figure 1) was therefore mapped manually by an experienced geologist. During the site mapping the team gathered 119 individual measurements, and following the site work and data analysis in rocscience Dips software, the stereonet shown in Figure 4(a) was obtained. The stereonet shown in Figure 4(b) is that from the digital rock mapping.

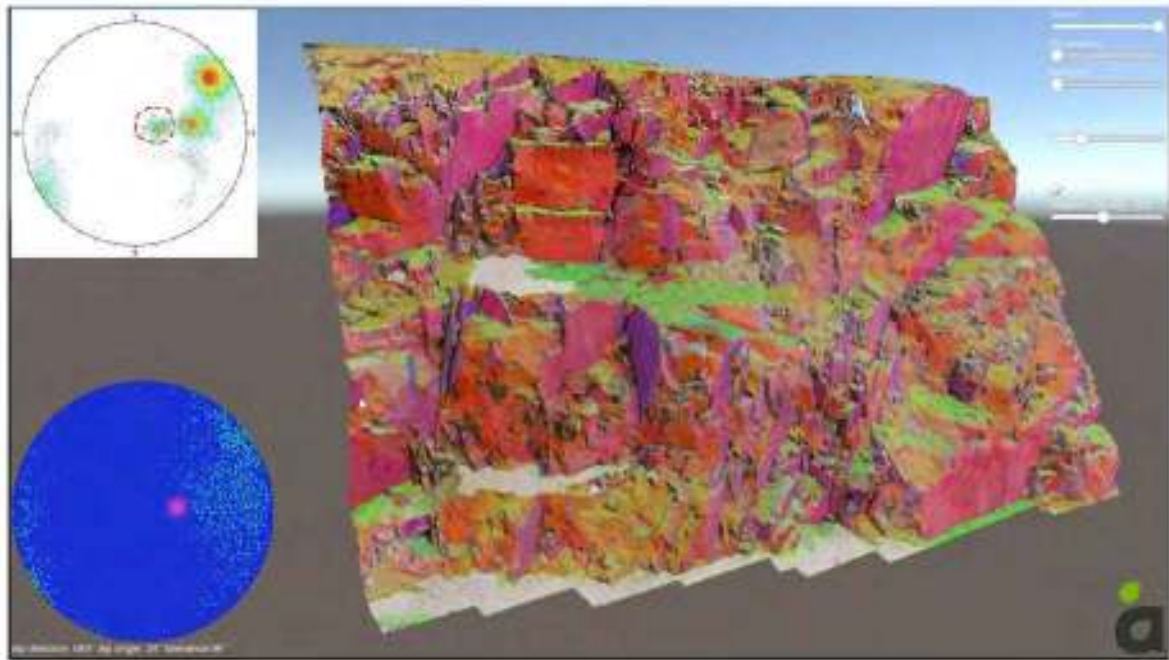


Figure 3: View of the Digital Rock Mapping program. The interactive stereonet of the DRM program in the bottom left shows the (pink) selected cluster of data displayed in white on the DRM model. The corresponding cluster is circled in red on the inset Stereonet produced from the DRM data using the Dips commercial software

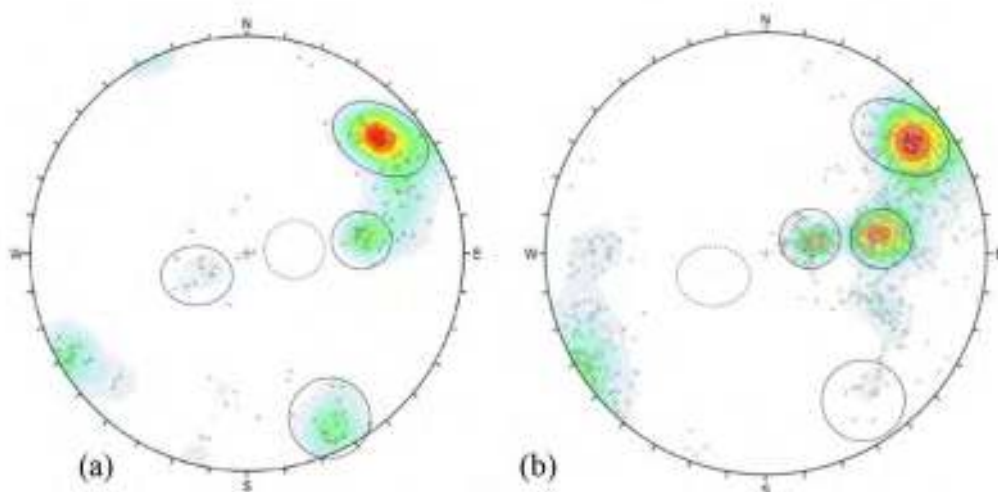


Figure 4: Stereonets from the manual data capture method (a) and the digital data capture method (b)



Figure 5: Increase in data collected and decrease in time when DRM is used over manual data collection

The digital mapping gathered significantly more data than the manual mapping, refer to Figure 5 and although very similar, the two stereonet in Figure 4 showed some clear differences (Gibbons et al, 2019).

The DRM stereonet showed one distinct near horizontal cluster (approximate dip/dip direction of 30/260) in addition to that from the manual data. The manual data indicated a near vertical cluster that is not so clear from the DRM results (approximate dip/dip direction of 80/320) the manual data also picked up a few measurements at approximately 30/072 which were not recorded by the DRM.

Upon further inspection, the near horizontal 30/260 cluster was shown to largely correlate to a portion of the quarry floor at the foot of the cliff and some of the ledges that did not relate to natural joint sets.

The near vertical cluster identified in the manual data set (80/320) was not so obviously identified in the digital data, there were however 14 measurements in the cluster from the manual data – representing approximately 12% of the data collected. The digital results showed 15 measurements within the same region of the stereonet, which represented approximately 2% of the data and therefore the digital stereonet does not indicate a strong cluster in that region. Viewing the model in Figure 3 it is clear there is a joint set in that region (shown by the near vertical dark purple planes). However due to the wider spacing and larger size of the planes compared with the narrower spacing of the often smaller 80/320 joint set, there are fewer measurements.

The 30/072 cluster in the manual results would need further inspection if this example were to be used for anything beyond a trial/development of the system. However, for the purposes of the development it reinforces the need for the DRM to supplement the manual mapping. The authors are not suggesting that the digital rock mapping can completely replace the manual mapping.

6 DESIGN IMPLICATIONS

With every joint and plane measured and recorded, a far greater understanding of the overall rock mass can be achieved. The interactive nature of the DRM then allows the designs to be tailored and detailed to specific portions of the cliff face, rather than the industry's current "blanket" approach. The greater level of certainty leads to less conservative and more targeted designs. The result being more efficient designs that save the clients both time and money.

One key advantage of undertaking the rock mapping remotely and in 3D, enables us to utilize the wide variety of other digital tools to communicate the specific details and risks to other stakeholders. The output from the digital rock mapping is typically imported into the virtual reality (VR) environment where geologists are now able to interact with a cliff face in a level of detail that has not previously been possible. Virtual reality provides the ability to undertake a virtual "site visit" to an area either not easily accessible or perhaps a repeat visit to an area.

The next step on from VR is augmented reality (AR) and it is in this sphere that technology has the potential to have significant influences on current work methods. Augmented reality provides the rock mapping data viewed in a way that it is superimposed on top of the real cliff view. Figure 6 shows the application of this at one of our New Zealand test sites.



Figure 6: Augmented Reality view of DRM on site

7 CONCLUSIONS

Only a couple of years ago the concept of mapping a rock face without necessarily even being in the same country seemed far too futuristic. By embracing the tools available and using them to develop more efficient, more accurate and more reliable processes, we have however been able to greatly improve the health and safety and provide more valuable results on projects. The digital rock mapping (DRM) tool is just one example and one application of the new methods that have very quickly become 'normal'. The DRM tool is also just one method to improve the digitization of engineering geological data gathering and analysis processes. With photogrammetry, VR and AR techniques already widely utilized, the digital rock mapping tool provides the link between the digital data that has been gathered, and the ultimate design or product that we need to communicate to clients and stakeholders. The software is however just another tool for the geologist to draw on, it will not replace manual mapping just yet but with increasing reliability it may do one day.

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Impact of Soil-Structure Interaction on the Seismic Design of Large LNG Tanks

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ABSTRACT

The rapid growth in LNG (Liquefied Natural Gas) supply market leads to an increasing demand for building large LNG tanks, many of which are located at areas with less favorable geotechnical conditions in seismically active regions. Different ground improvement techniques (e.g. Deep Cement Mixing, Stone Columns) are usually considered as alternatives to deep foundations to support LNG tanks, and the seismic performance of LNG tanks supported by improved ground during strong ground shakings is considered crucial and can govern the design. This paper presents the application of advanced numerical simulation techniques to study the impact of Dynamic Soil-Structure Interaction (DSSI) on the seismic behaviour of large LNG tanks sitting on an improved ground, where a non-linear three-dimensional finite element model employing LS-DYNA is developed to perform time-history analysis. Employing the advanced numerical simulation techniques can overcome the need for unrealistic simplified assumptions as adopted in conventional approaches. This enables a more realistic simulation of actual behaviour of large LNG tanks subjected to earthquake loading which results in considerable cost savings while ensuring the design safety.

1 INTRODUCTION

The LNG (Liquefied Natural Gas) supply market has doubled in the last decade and it is anticipated that the next decade will see further growth, and this leads to an increasing demand for smart and affordable storage solutions (i.e. LNG tanks). To achieve economies of scale throughout the LNG chain, the size of liquefaction plants and the cargo size of LNG vessels have steadily increased requiring an increase in the size of LNG storage tanks (Peiris et al, 2006).

The LNG tanks are commonly located at offloading terminals prior to re-gasification and distribution by pipeline. This means that the terminals and LNG tanks are necessarily at coastal sites often with less favorable ground conditions. Different ground improvement techniques (e.g. Deep Cement Mixing, Stone Columns) are usually considered as alternatives to deep foundations to support LNG tank for satisfying the foundation design requirements in terms of bearing capacity and settlement.

In the view of hazardous nature of the product, the seismic design of the LNG tanks is critically important, and design follows the similar philosophy to nuclear power plants using the concepts of Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE) scenarios with no leakage permitted at SSE (Willford et al, 2010). The critical review of the seismic design criteria for LNG plants were presented by Lubkowski et al. (2006).

This paper presents the application of advanced numerical simulation techniques to study the impact of Dynamic Soil-Structure Interaction (DSSI) on the seismic behaviour of large LNG tanks sitting on an improved ground. The paper is based on seismic design experience of a real project located in an area with high seismicity, where advanced non-linear three-dimensional finite element models were developed and used to perform time-history analysis.

2 GROUND CONDITIONS

The in-situ ground under the LNG tank of the selected case study generally consists of 1m of sand fill, which overlay 3m of organic soil, which in turn overlay 10-15m of silty-sandy alluvium deposit with coarse sands towards the bottom. Claystone bedrock, which is underlain by weathered claystone formation of about 5-6m thick, is at about 22-27m depth. The ground water found at +0.5m. The simplified in-situ ground model is shown in Figure 1.

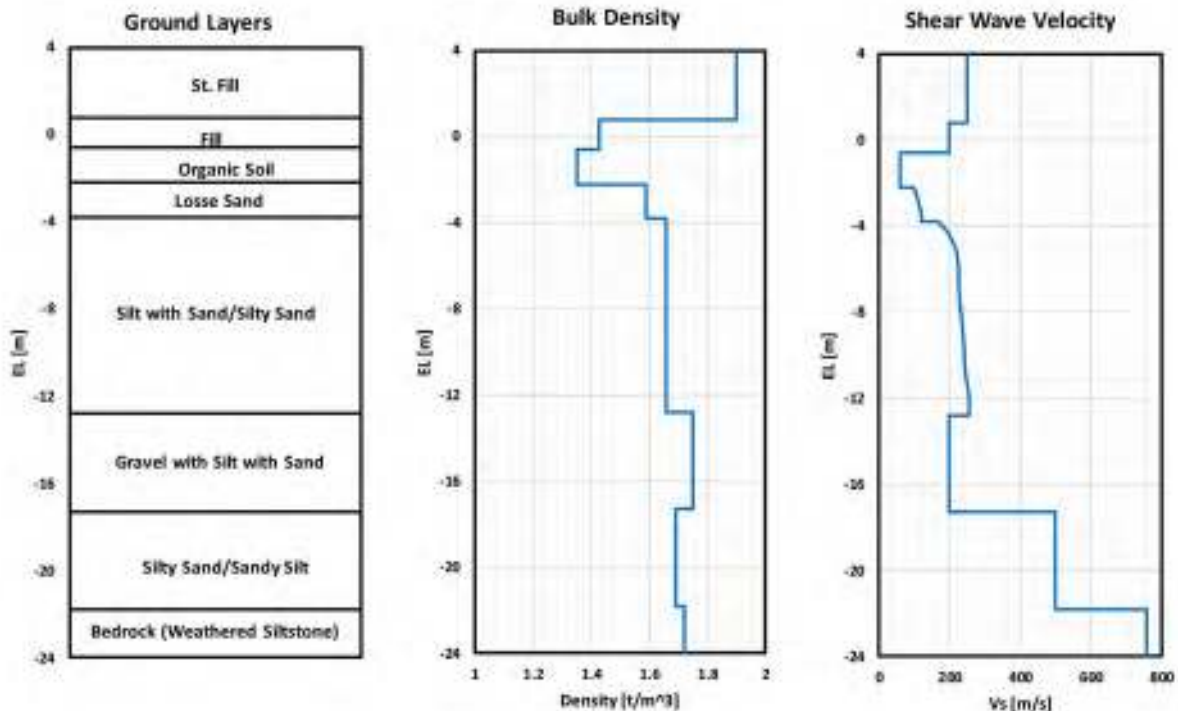


Figure 1: Simplified in-situ ground model

The organic soil having undesirable mechanical properties, which can undergo excessive settlement subject to surcharge loadings, was suggested to be 100% replaced with structural fill materials. Furthermore, the in-situ surface elevation was planned to be raised by 3m using the same structural fill materials.

The detailed liquefaction assessment conducted by Arup suggested that the silty-sandy layer has liquefaction potential under the design earthquake, and different ground treatment options for the purpose of liquefaction mitigation were investigated. Employing different ground improvement techniques (i.e. pile foundations, Deep Cement Mixing, Stone Columns) and configurations, in addition to their suitability for liquefaction mitigation, can alter the dynamic properties of the ground, mainly in terms of stiffness and damping, and this can influence the seismic response of the tank subjected to design earthquakes.

3 DESIGN EARTHQUAKE GROUND MOTIONS

In order to conduct a three-dimensional DSSI analysis in LS-DYNA, suitable ground motion time-history records in two horizontal directions and one vertical direction (orthogonal to each other) are required.

The ground motion records were selected to be compatible with the expected characteristics of the ground motion at the site. These characteristics were derived based on site specific Probabilistic Seismic Hazard Assessment (PSHA) done by Arup which defined the Uniform Hazard Response Spectra (UHRS) and the magnitude/distance combinations of the earthquakes that dominantly contribute to the hazard at the site.

The time-history records were selected from database of measured strong ground motion records that match the de-aggregated earthquake magnitude and distance combinations that contribute most

significantly to the hazard at the site. The selected time-history records were modified by the spectral matching technique to match the uniform hazard response spectrum (UHRS) for the site. Without modification, the response spectra of the selected time histories in most cases will not closely match the target design response spectrum.

The computer program RSPMatch2005 (Hancock et al., 2006) was used to match the time-history records. RSPMatch adopts a time-domain approach to modify the recorded ground motions to match the specified target response spectra by adding wave packets of various periods in discrete durations. These wave packets are added to the recorded time-history within small and discontinuous time windows where there is significant amplitude of motion of the same period. This method reduces the modification of the ground motion to a minimum and is considered to preserve many of the natural features of the ground motion, while changing it subtly to match the uniform hazard response spectra.

As an example, Figure 2 shows the UHRS and the spectrally matched response spectra for the selected ground motion records for horizontal and vertical directions, respectively.

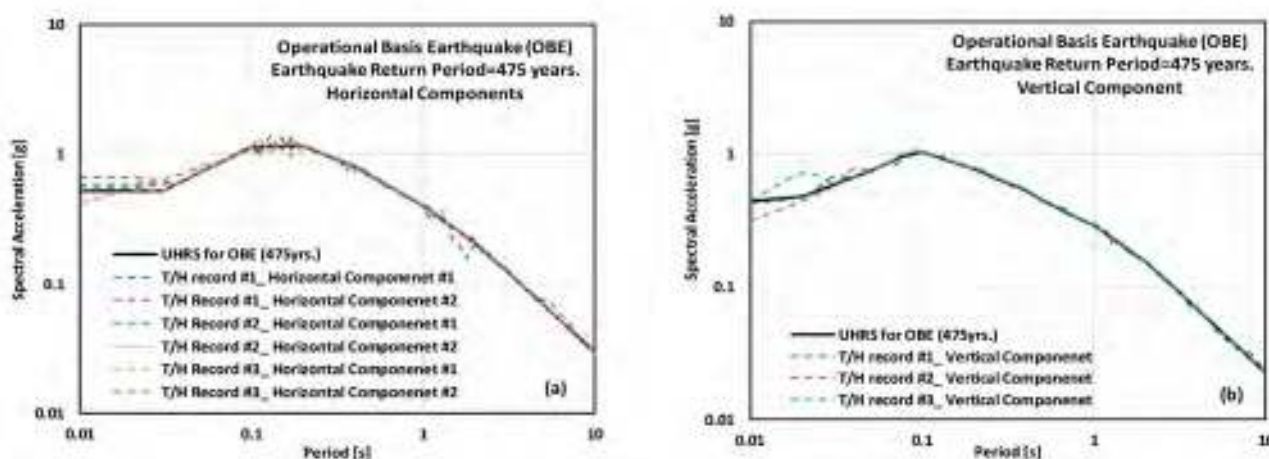


Figure 2: Example of spectrally matched time-history records for DSSI analysis:
(a) Horizontal components; (b) Vertical component

4 DEVELOPMENTS OF NUMERICAL MODEL FOR DSSI ANALYSIS

A number of computer programs were developed to examine earthquake DSSI problems during the 1970's (e.g. FLUSH, SASSI, CLASSI etc.) to address the design of nuclear power facilities in the US, (Lubkowski et al, 2004), among which SASSI has been widely used for the DSSI analysis of tanks. In the recent years, the development of advanced computational tools and numerical methods allows the design engineer to undertake direct nonlinear analysis and overcome the limitations of the conventional approaches.

Bolisetti et al. (2015) presented a comparison of the linear frequency-domain code, SASSI, and the time-domain finite-element code, LS-DYNA, for DSSI analysis. In the direct method (i.e. LS-DYNA), the whole soil-structure system is analysed in a single step thereby circumventing the use of superposition which is extensively used in traditional SSI analysis methods (including SASSI) and is restricted to linear analyses. This enables a more realistic simulation with the use of nonlinear material models for the soil and structure, and contact models that simulate separation and sliding at the foundation-soil interface.

In this study, the general-purpose finite element program LS-DYNA was used for DSSI analysis. This program is an explicit dynamic code that runs in the time domain and incorporates both structural and soil non-linearity. It is equipped with a large number of material models that can be used to model the soil and structure, and several contact algorithms that can be used to model the foundation-soil interface. Arup has been the co-developer of the software and many of LS-DYNA's capabilities for civil engineering were developed by Arup (Willford et al., 2010). This software has been successfully used by Arup over many years for a wide range of foundation and soil structure interaction problems (i.e. Lubkowski et al., 2000).

The developed mode for DSSI analysis allows the following effects to be directly taken into account:

- The propagation of earthquake waves from bedrock to ground surface;
- The increase in stiffness of the soil due to ground improvement;
- The stiffening effect of the structure on the soil;
- Degradation of soil due to the ground motions and self-weight of the tank;
- The impulsive and convective forces of the LNG in response to earthquake loading and the interaction with the foundation soils;
- The radiation of energy away from the structure (radiation damping).

4.1 Modelling of superstructure

The finite element model of the superstructure was developed for the DSSI analysis. The above-ground structure consists of an inner steel tank inside a secondary concrete outer tank. The inner tank material is 9% Nickel Steel with wall thickness of ~10-40mm, while the outer tank is made of prestressed reinforced concrete with wall thickness of ~700mm, roof thickness of ~400mm, and base thickness of 500-1000mm. Different tank sizes were investigated with outer tank diameter of ~80-90m and height of ~45-60m. In the numerical model, both the inner and outer tanks were modelled using shell elements (see Figure 3).

As explained by Jaiswal et al. (2007), seismic analysis of liquid-containing tanks differs from buildings (e.g. Hokmabadi et al., 2014) in two ways: first, during seismic excitation, liquid inside the tank exerts hydrodynamic force on tank walls and base. Second, liquid-containing tanks are generally less ductile and have lower redundancy as compared to buildings.

4.2 Modelling of the fluid (LNG) contents of the tank

The hydrodynamic response of the fluid (LNG) contents of the tank are usually evaluated using mechanical analog in the form of spring-mass system, which simulates the two main vibration modes of the fluid inside the tank, including (Malhotra, 2000):

- Impulsive mode (rigid movement of the liquid)
- Convective model (sloshing mode).

The hydrodynamic properties of lumped mass are expressed in terms of effective mass, height, period (stiffness), and damping. There are several approaches to derive such parameters as reflected in different codes. The common seismic codes and guidelines are compared by Jaiswal et al. (2007). Gibson et al. (2015) evaluated the lumped mass analogue model with the actual fluid structural interaction simulation for storage tanks seismic assessment.

In this study, the method described in API650 was used to determine the hydrodynamic properties of the impulsive and first convective modes. The second convective and first tank-liquid vertical breathing modes were calculated according to NZSEE (2008).

In the numerical model, the impulsive and convective masses were attached to the inner tank wall with a spring and damper in both the X and Y directions, and the vertical mass is connected to inner tank base with a spring and damper in Z direction. The simulation of the tanks and the connected hydrodynamic lumped masses in LS-DYNA are shown in Figure 3.

4.3 Modelling of subsoil

The soil nonlinear behaviour under cyclic loadings was considered by defining the soil shear modulus degradation curves for each soil type. The shear modulus degradation curves represent the reduction in soil shear modulus (G/G_0) as a function of soil shear strain. The soil model consisted of 8-noded solid elements using the non-linear soil material model MAT_HYSTERETIC_SOIL. This model captures the hysteresis of the soil under cyclic loading.

The merit of using nonlinear soil models over the simplified equivalent linear methods for DSSI analysis are discussed in the literature (i.e. Fatahi and Tabatabaiefar 2013; Xu and Fatahi 2019). The nonlinear soil models can capture the cyclic nonlinear behaviour of the soil more accurately, while in the

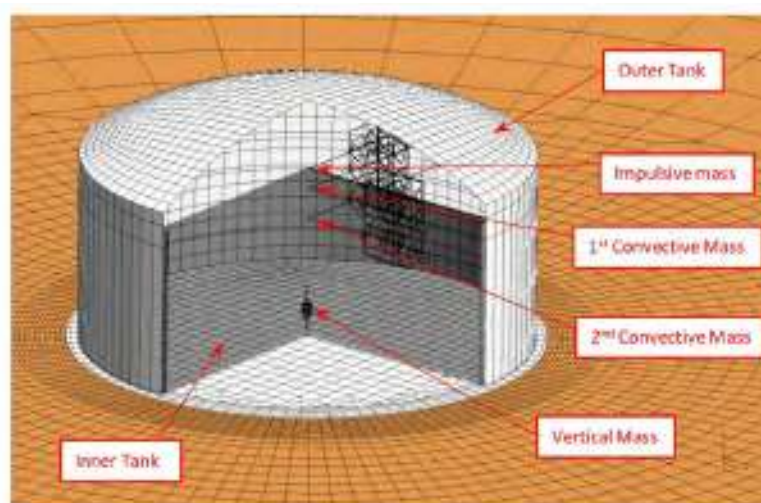


Figure 3: LNG tank and hydrodynamic lumped mass simulation in LS-DYNA for DSSI analysis – with cut-out

equivalent linear methods, the strain-dependent modulus and damping functions are only taken into account in an average sense to approximate the soil nonlinearity.

The overall dimensions of the model were chosen as sufficiently large ensuring the free-field conditions close to the edge of the model as shown in Figure 4. The ground motion time-history records were applied at the model base as a tri-axial earthquake excitation along the three orthogonal directions (X, Y and Z axes).

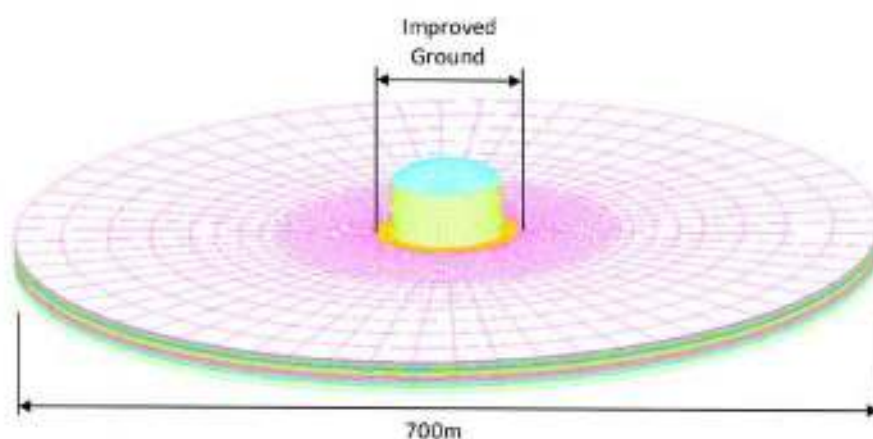


Figure 4: Soil block dimensions – outer zone and central improved zone

Non-reflecting boundary conditions were considered at the bedrock using the method developed by Lysmer (1978). This lower artificial boundary condition simulates an infinite half space, where discrete viscous damper elements were modelled at the bedrock along each direction with properties calculated from the characteristics of the bedrock of the site. Each soil layer perimeter node set is constrained, so as not to allow relative displacement. Each constraint is also globally restrained from rotation.

4.4 Modelling of ground improvement

The ground improvement scheme involved 100% replacement of Organic Soil and Fill layers with structural fill materials, while the layers below will be treated with ground improvement inclusions at certain replacement ratio down to the top of the competent soil layer.

In the DSSI analysis, the structural fill properties were adopted for Fill and Organic Soil layers for the improved ground. Below the Organic Soil layer, the individual ground improvement inclusions were modelled as shown in Figure 5.

The explicit modelling of ground treatment inclusions, rather than the simplified approach of using homogenised (equivalent) properties for the improved layers, results in more accurate estimation of ground responses under earthquake excitations and allows for detailing and further design optimisation.

5 NUMERICAL ANALYSIS RESULTS

The results of DSSI analysis can be extracted in terms of accelerations, loads/deformations, stresses/strains, etc. Some of the key outputs that are crucial for the design of LNG tanks are discussed below.

5.1 Hydrodynamic lumped mass accelerations

The hydrodynamic lumped mass time-history response and peak accelerations can be directly extracted from DSSI analysis. This includes response of the impulsive mass, the convective masses, and the vertical mass. Figure 6 for instance shows the acceleration experienced by the impulsive mass under the imposed time-history records in the horizontal direction. The estimated responses from the numerical simulation can then be compared with the design code limitations for use in the design of the tanks.

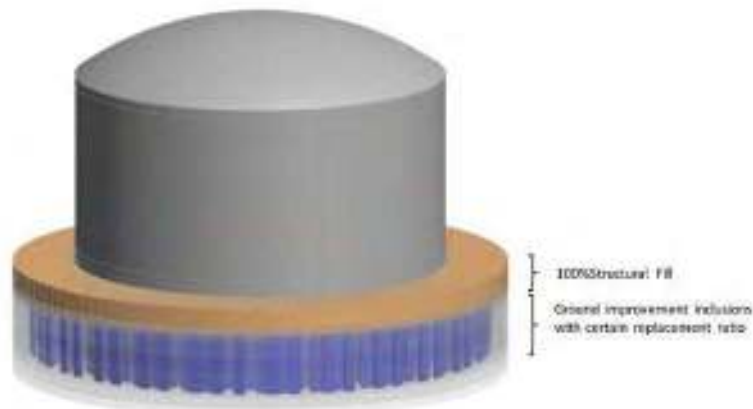


Figure 5: Numerical simulation of ground improvement under LNG tank

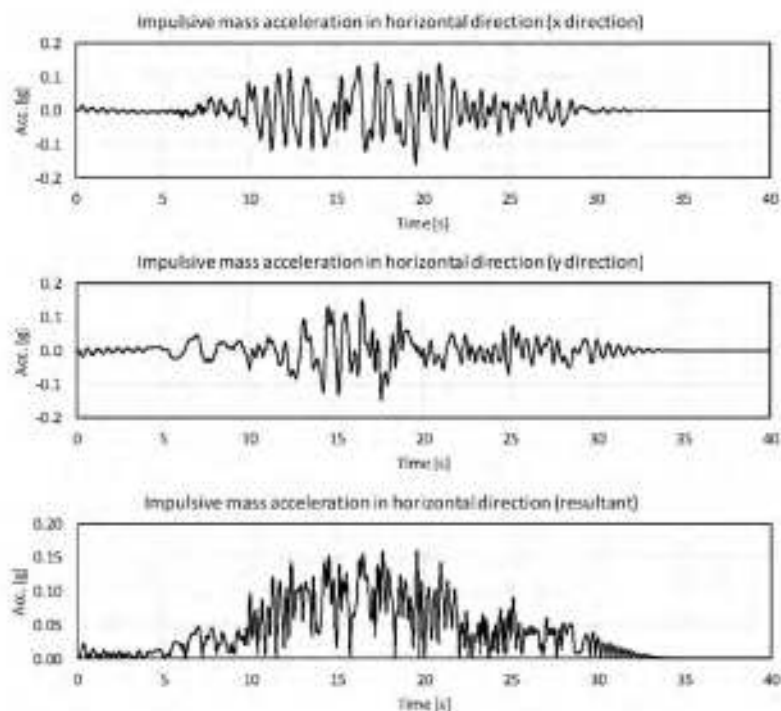


Figure 6: Sample of results – Impulsive mass acceleration responses in horizontal direction

5.2 Inner tank and outer tank accelerations

The Inner/Outer tanks peak accelerations can be extracted at required locations (i.e. along the walls and roof) from DSSI analysis for the structural design of the tanks. Figure 7, for instance, shows the acceleration responses of the outer tank roof in the horizontal direction.

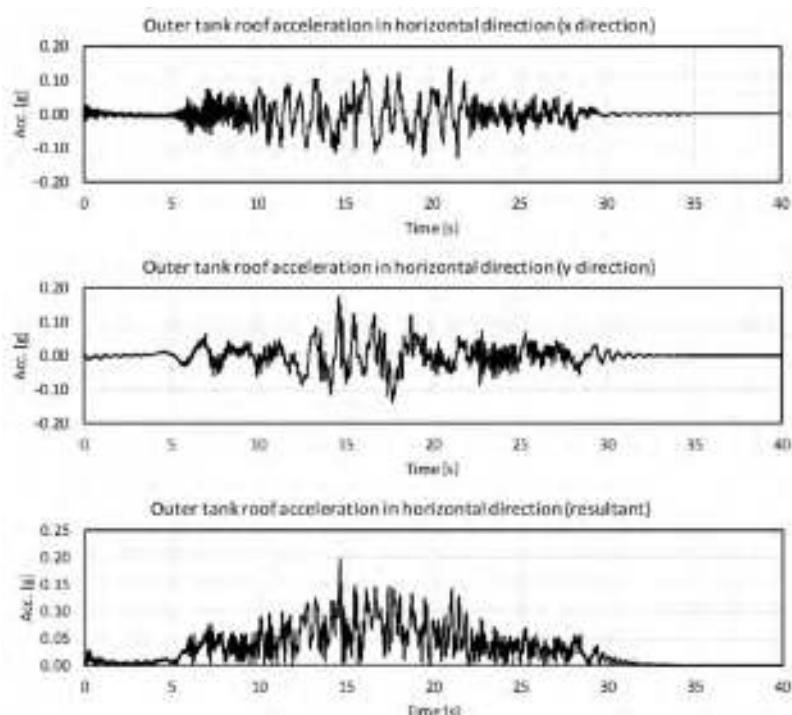


Figure 7: Sample of results – Outer tank roof acceleration responses in horizontal direction

5.3 Inner tank sliding and uplift assessment

The developed DSSI analysis, by defining an appropriate joint element between the inner tank and outer tank, can be used for checking the inner tank sliding and uplift over the outer tank concrete base slab during the seismic excitations. This is an important design requirement in the project. The developed advanced DSSI analysis can provide a more accurate assessment in the time domain, rather than just comparing the maximums/minimums, to ensure the design requirement is satisfied and allow a more realistic assessment in terms of the need for anchoring or other equivalent measures.

Figure 8 for instance shows the horizontal/vertical force ratio of the inner tank over the outer tank concrete base slab for the inner tank sliding check. This ratio can then be checked against the available sliding resistance and the required minimum factor of safety.

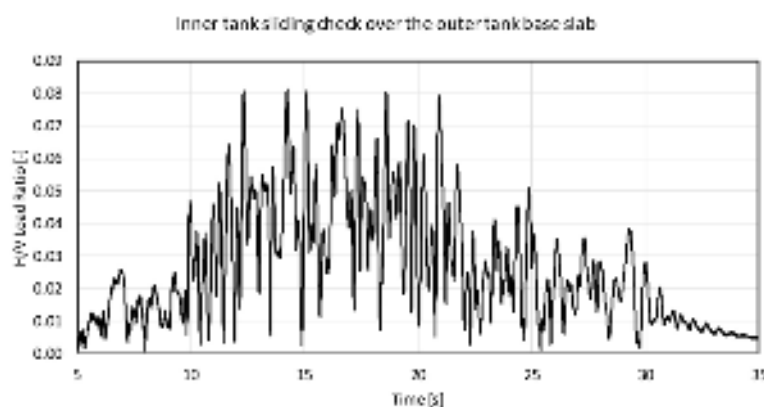


Figure 8: Sample of results – Horizontal/vertical force ratio of the inner tank over the outer tank concrete base slab for the inner tank sliding check

5.4 Foundation and ground improvement design

The foundation loads from DSSI analysis can be extracted for the foundation design. Moreover, the results of DSSI analysis can be used to assess the maximum stress/strain of the in-situ soils and ground improvement inclusions (stress distribution), and piles (if used) under seismic loadings.

Figure 9 shows the experienced shear strain in the ground improvement inclusions under the design earthquake.

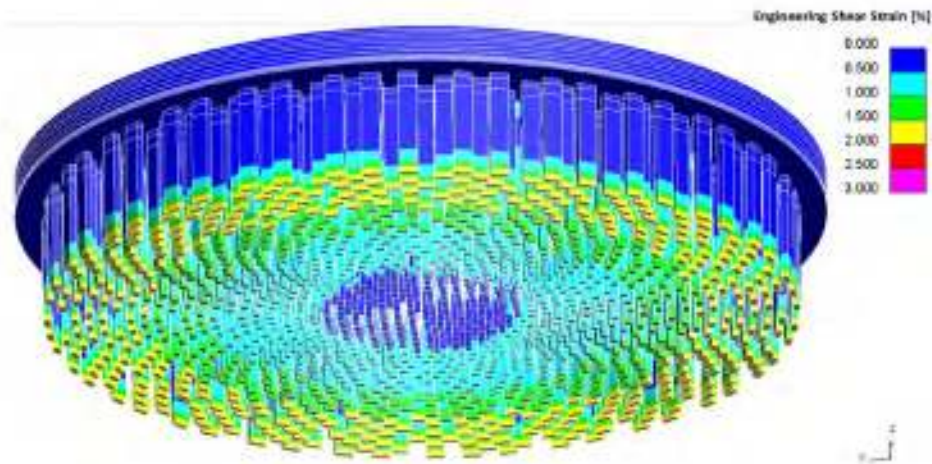


Figure 9: Sample of results – Experienced shear strain in the ground improvement inclusions under the design earthquake

6 CONCLUSIONS

The application of advanced numerical simulation techniques to study the impact of Dynamic Soil-Structure Interaction (DSSI) on the seismic behaviour of large LNG tanks sitting on an improved ground is presented. For this purpose, a non-linear three-dimensional finite element model employing LS-DYNA is developed to perform time-history analysis. The impulsive and convective modes of the liquid inside the tank are simulated. Hysteretic behaviour of the soil is implemented to account for the degradation of soil shear modulus as a function of cyclic shear strain. Appropriate boundary conditions are adopted to ensure capturing of free-field motion in the far-field as well as radiation damping. Suitable contact models are defined to simulate the possible separation (uplift) and sliding of the tanks. Moreover, the ground treatment inclusions are modelled explicitly, rather than the simplified approach of using homogenised (equivalent) properties for the improved layers, which results in more accurate estimation of ground responses under earthquake excitations and allows for detailing and further design optimisation.

It was shown that the advanced numerical simulation techniques can overcome the need for unrealistic simplified assumptions as adopted in conventional approaches. This enables a more realistic simulation of the actual behaviour of large LNG tanks subjected to earthquake loading, and previous project experiences show that this can result in considerable cost savings without compromising design safety.

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A ‘whole-of-life’ approach to project 3D ground-models: City Rail Link, New Zealand

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ABSTRACT

The City Rail Link Project in Auckland has been designed and consented over the last 5 years and enabling works for the project are currently under construction. This paper reviews the introduction and progressive use of 3D geology modelling over the course of the project. Initial 3D geology modelling was experimental and internally focussed. As modelling capabilities became better developed the models were shared within the design team and with other stakeholders. When software for managing models and associated factual data was introduced, the models became an integral part of the tender for the CRL main works. The use of 3D geology has not removed the requirement for 2D interpretations, but rather is used alongside the 2D sections as an aide to communication. Investment in 3D geology enables export to a range of other analytical software and encourages the development of automated design tools with geology as one of its inputs. We observe that 3D geology has also encouraged maintenance of a whole-of-life 3D geology model (or models) for a project, overcoming traditional barriers to sharing of geological interpretations across the procurement process.

1 INTRODUCTION

1.1 Geological Models

Three dimensional representations of geological models have been used since the early days of geology, valued for their unique ability to convey meaning and information about processes and geometry to a wide audience. However, 3D block models are quite difficult to prepare, requiring a combination of artistic ability and scientific knowledge. Most practitioners have illustrated their models with a combination of 2D sections, plans and text. These are easier to prepare, but require an audience with relevant experience or training to make sense of the models.

Similarly, digital approaches to 3D geology are not new, with many software packages having offered a range of functionality for decades. For many years these were very expensive, and therefore mainly used in the petroleum and mining exploration industries.

In this paper we report on the adoption of 3D geology modelling on the City Rail Link (CRL) project, using relatively inexpensive software, relying on increasing availability of 3D-ready digital data from ground investigations, and leveraging the increasing use of modelling and visualisations across most engineering and associated disciplines. The brief case study presented shows the progressive adoption of 3D geology, the benefits realised, and problems encountered. Finally, we consider the relevance of 3D modelling for Hong Kong infrastructure projects.

The project lies mainly in the weak rocks of the Waitemata Group (turbidite sequence of alternating sandstones and siltstones). Parts of the alignment also cut through complex deposits of several Auckland volcanoes (basalt lava and ash), and into Pleistocene alluvium and Holocene marine sediments (Figure 1).



1.3 Design & Consents and supporting ground investigations

Design and consenting efforts were supported by the ongoing development and evolution of a ground model and a supporting ground investigations database. The database was established with extensive research for desk study sources (ultimately finding more than 1300 historic holes in the project corridor) and a series of five project-specific investigations [Preliminary (Stage 1, 2010), main investigation (Stage 2, 2012-2013), supplementary investigations in 2015 (Stage 3) and 2016 (Stage 4). Pre-construction investigations were finalized with tender-stage investigations in 2017-18 (Stage 5)].

Ground and ground risk interpretation and documentation have followed a standard model leading up to procurement (desk study, factual reporting, geotechnical interpretative reporting, with

geotechnical baseline reports and data reports for procurement). Consenting was supported by a range of assessments of environmental effects (for groundwater, settlement, vibration, contamination, and other topics).

2 Ground Models in 2D and 3D

2.1 2D Interpretation: Long and Cross Sections

Representation of the ground model in 2D profiles (longitudinal and cross sections) has been a key component of the ground model from the start and has remained so even as 3D models have been introduced. 2D sections have a well-established set of procedures for quality assurance (QA), representation of uncertainty, and use for geotechnical analysis, design and during construction. As is further discussed below, we find that 3D models, even after several iterations, do not necessarily reach the refinement of our fully developed 2D sections. We do find that our methodology for developing 2D sections is improved through the use of 3D, by using “3D-assisted” sections to help resolve early geometric difficulties and by registering 2D sections in the 3D model during QA to ensure consistency between long and cross sections. Despite the higher resolution associated with 2D long section production, in complex geological settings, particularly associated with basalt flows, 2D long sections are not as effective at portraying the overall distribution of materials, even when supplemented with cross sections. This is where the benefits of a 3D representation become apparent, particularly in communicating geological risk to the tenderers.

2.2 Progressive adoption of 3D geology models

3D geology modelling was introduced as an experimental in-house tool during concept-design to help communicate complex ground in several discrete areas of the alignment. Communication between the geotechnical team and designers was noticeably improved. Distribution of models was wider than expected as non-traditional users of ground models bought-in to a visual method of communication. Further distribution was aided by improvements in interoperability and multi-directional transfers of 3D models between engineering disciplines and across multiple platforms. Issues arose around the level-of-development of models, uncertainty communication (and visualisation of uncertainty), and currency of models that were under an iterative process of development. These problems were largely resolved by improved QA, document management and upgrades and developments by our software vendor (Rose et al. 2018).

Software upgrades during the period have included migration from Leapfrog GEO (based on mining exploration workflows) to Leapfrog Works (which supports a better range of infrastructure-friendly capabilities), and more recently, the adoption of Leapfrog Central, which manages models (and subsets of models), provides facilities for sharing (or access restrictions), and tools for documentation of model versions, QA and discussion. Table 1 documents the chronological development of the use of 3D models on the CRL project, commencing with internal use by the designers in 2014 on Contracts 1 and 2. For contract 5 and 6, the 3D model was used to develop the 2D long sections and was provided to tenderers within a non-editable viewer. Feedback at that time from tenderers was that the viewer was very useful on communicating the geological conditions and risks. For the main tunnels and stations of Contract 3, Leapfrog Central was used to host the client tender model as well as the models that were developed and enhanced by each of the tenderers. One of the main advantages of this approach was that tenderers did not need to expend time and effort on processing geotechnical data but could instead focus their efforts on revised interpretation and risk mitigation. This approach also enabled a transparent evaluation of the geotechnical risk profile adopted by each tenderer as the client had full access to the developed 3-D models.

Table 1: Use of 3D Geology in CRL Contracts (in chronological order)

Contract	Works Description	Procurement type	Tender Geology provided	Software (3D Geology)
Contract 1 (2014)	Britomart underpinning	ECI Lump Sum	2D sections in GBR (equivalent)	3d model internal to PTA, for visualization of complex geology (Leapfrog GEO)
Contract 2 (2014)	Albert St cut and cover & stormwater realignment	ECI Lump Sum		
Contract 6 (2016)	Mt Eden Stormwater replacement	Lump Sum	2D sections in GBR; and non-editable 3D model for information only	Leapfrog Works and Leapfrog Viewer (and exports to hydrogeology and other modelling software)
Contract 5 (2017)	NAL connections	Lump Sum		
Contract 3 (2019)	Main Works (Aotea – Eden, including tunnels and stations)	Competitive Alliance	2D reference section, 3d model and supporting factual data provided	Leapfrog Works (as above) Leapfrog Central for managing PTA and tenderers' independent models and appreciation of ground risk

Contracts 1, 2 and 6 are all presently in construction. Use of 3D geology models in C1 and C6 are summarized below.

2.3 Construction (C1): Observational Approach to the Pre-Holocene Channel at Britomart

At Britomart Station, where existing buildings are to be underpinned as part of tunnel constructions works, geological interpretation was challenging due to limited opportunity to carry out investigations prior to construction. The geological interpretation, based on some historic GI and limited project-specific GI, showed the site was affected by a paleo-channel that had eroded into older sediment and rock, and been infilled by soft Holocene marine mud. The entire site was on reclaimed land – part of the widespread reclamations carried out along this part of the coast in the late 1800s – early 1900s. Although the presence of the channel was known, there were uncertainties about its exact extents: depth, width, and axial position. For this reason, construction stages relied on an observational approach (pre-drills for diaphragm wall panels) to define actual site conditions, and final depths for panels and associated piles. Figure 2, shows the original panel designs with pre-drills, and the revised geological model.

Construction phase drilling proved that the channel was slightly east of the design-stage interpretation, and the panel and pile depths therefore required revision. The erosional extent of a paleochannel can be challenging to represent in 2D and much better communication was achieved using the 3D model.

2.4 C6 Complex Ground between Two Volcanoes

At the CRL southern portal area, the tunnels surface from bedrock into an area dominated by ash and lava from the Domain and Mt Eden volcanoes. The existing North Auckland Railway Line runs along the relatively flat land that lies at the junction of these materials. Proposed works include east and west-facing connections to the NAL, and replacement of a stormwater main that lies in the path of the proposed CRL infrastructure.

The area has a history of adverse ground conditions and associated engineering difficulties. When the existing stormwater was being constructed the first tunneling attempt unexpectedly encountered basalt (in a TBM not designed for hard rock), and the second attempt encountered unanticipated very soft ground (but managed to negotiate it).

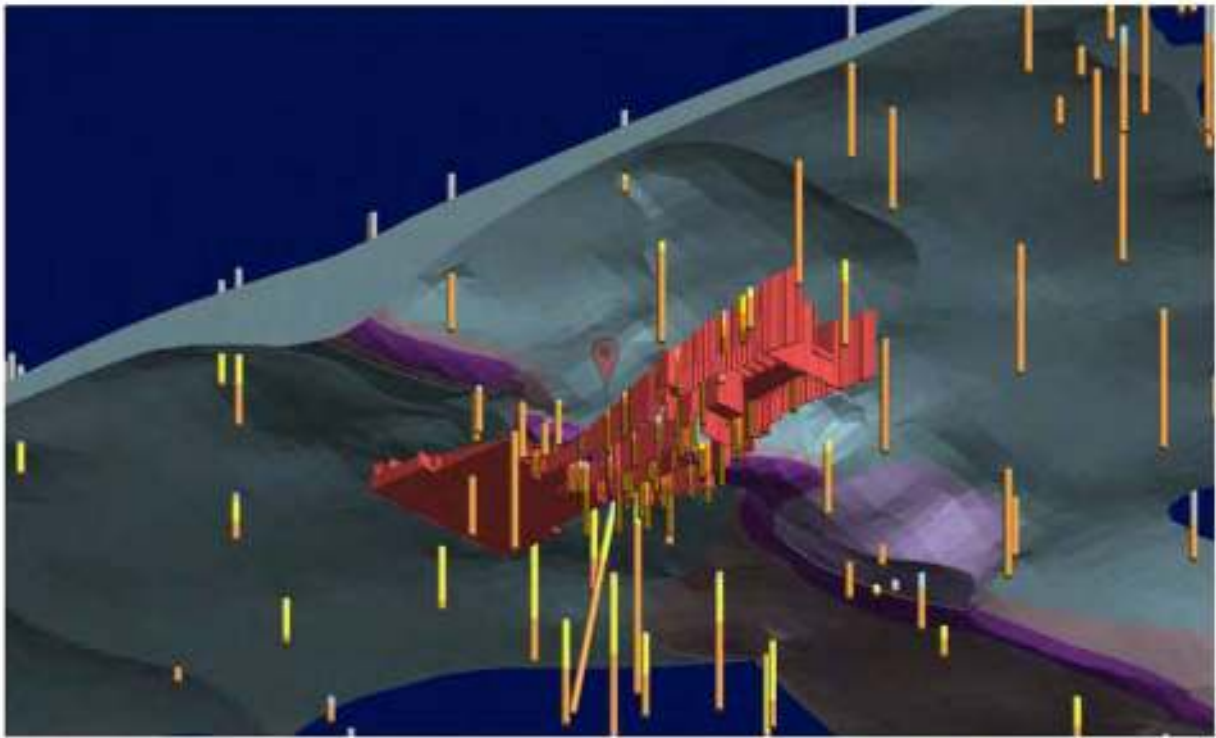


Figure 2: Observational approach at Britomart – interpretation of meandering paleo-channel (from below): Blue is marine mud, purple is a basal channel-lag deposit, red is the foundation design.

Ground investigations for CRL thoroughly investigated ground risks and used 3D modelling to better communicate the new interpretation to tenderers. The 2D approach (using numerous long and cross sections) was considered inadequate by itself as the engineering geometry was complex, and abrupt lateral variations in lithologies meant that a large number of sections was required to communicate the geological setting. The effectiveness of the geological model was greatly enhanced by incorporating a ‘painted’ 3D visualization of the proposed engineering works (Figure 3), in which tunnels and other structures were colour-coded with the symbology of the lithology they intersected.

At the time of writing, the replacement stormwater works are nearing completion. Tunneling has proceeded (with a machine capable of handling basalt) largely under the basalt but just clipping the

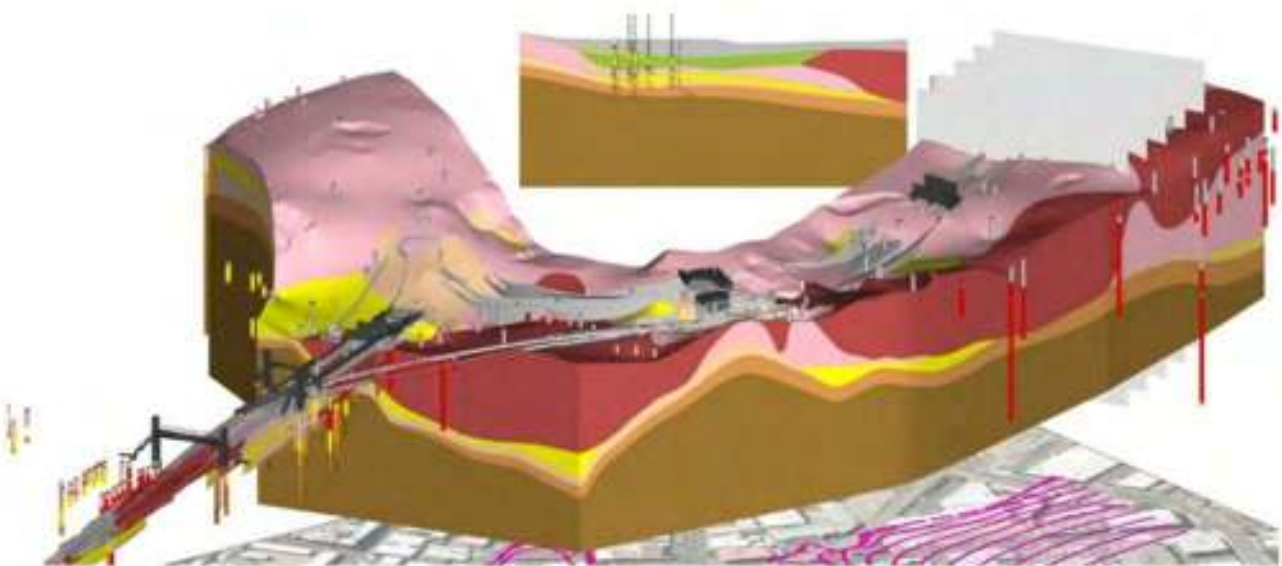


Figure 3: Mt Eden lava (red) overlies Domain Ash (pink) with complex sediments at the boundary (Rose et al. 2018)

base of basalt in one location. Shafts were designed to handle very soft ground and basalt. The soft ground shaft was completed successfully, while the basalt shaft is experiencing some slow progress due to variations in basalt thickness. Data from the (contractor's) shaft excavations and tunnels has been incorporated into the (client's) 3D geology model to inform contractual communication relating to differing ground conditions. The ability to show construction stage ground observations in a view with tender stage (3D) ground interpretation is a powerful means of communication when addressing unexpected ground conditions.

2.5 Benefits of Collaborative Procurement and beyond

More recently, 3D models have become integral to procurement for the main works (Contract 3) as a cloud-based model server became available (Leapfrog Central), with facilities to manage multiple versions of, and control access to, 3D models. This interactive 3-D model facilitated the required interaction with tenderers on ground risk.

During the procurement phase, CRL placed high value on design and construction mitigation measures proposed to address ground risks, as is appropriate for a major underground project. In order to achieve alignment on the level of geotechnical risks, an Alliance Geotechnical Baseline Report (AGBR) process was developed, which involved multiple alignment workshops and submissions that were all facilitated through the shared development of the cloud-based 3D model.

The 3D model was submitted for CRL review prior to submission of the tender in order to develop a common understanding of the ground risks addressed by the tender.

The AGBR and 3D model development process was very successful and resulted in both tenderers having a very similar ground risk profile. This meant the tender process was not skewed by one tenderer having a higher project risk profile. Neither tender included unmanageable ground risks.

2.6 Relevance to Hong Kong

With its complex geology and dense urban development, Hong Kong is particularly suited to the use of 3D ground models for major engineering projects. Moreover, Hong Kong has a remarkable library of historic ground investigation reports (CEDD's GIU) many of which include supporting digital information in AGS data transfer format (Lai et al. 2019).

Compared to New Zealand's digital ground data, Hong Kong has a more extensive digital database as a result of having acquired AGS data for more than 25 years. But the data quality is relatively poor, especially for older data. New Zealand has started compiling its national geotechnical database relatively recently, but records, where available, are generally high quality. Both governments have recognized the value of historic factual ground investigation data and are investing in improving and extending their databases.

Our work highlights that there is still a large gap between these regional factual datasets and the information required to support the design and procurement of major infrastructure. We show the benefit of developing and sharing cleaned GI factual data and associated developed 3D interpretations. A corollary of this is the need to extend national or regional GI databases from records of original factual data to incorporate value-added information that is typically developed for design and procurement of major projects.

3 CONCLUSIONS

Concept and Reference Design for the City Rail Link Project have been in progress for more than seven years. During that time, capabilities for 3D modelling of geology have been rapidly developing, matching similar developments in visualization for other disciplines. These technologies have been used to add value internally, with the client and other stakeholders, and to help tenderers rapidly get up to speed with ground conditions and reach a common understanding of ground risk.

The availability of 3D models, and software that enables separation (and comparison) of client-side and contractor-side interpretations has also resulted in significantly better communication of ground conditions between client and contractor than would have been possible with 2D geology alone. In areas

where the complexity of project or ground risk warrant it, 3D geology modelling represents current best practice in developing a common understanding of ground risk (Ireland et al. 2014).

The use of 3D geology, particularly within the Alliance contract model, has helped break down traditional barriers between sharing of geological interpretations between client and contractor, and has resulted in a successful tender process where ground risks are identified and expressly mitigated.

National or regional databases of historic factual ground investigation data in digital formats are a key component in being able to build 3D ground models. However, incompleteness and quality issues with such data make working with this data difficult. This can be addressed by extending national and regional databases to include cleaned and value-added records and interpretations.

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AGS Data – Subsurface Data in Digital Format in Hong Kong

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ABSTRACT

Obtaining subsurface data through ground investigation (GI) and laboratory testing (LT) information is a critical part of most engineering projects, especially those related to infrastructure development and geotechnical studies. AGS data format which is published by the Association of Geotechnical and Geo-environmental Specialists (AGS) has become one of the contract requirements in the GI and LT term contracts administered by Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) since 1993. Most Government GI contracts have also adopted similar requirements for AGS data. When Government departments submit GI and LT reports testing reports to the GEO's Geotechnical Information Unit (GIU), they also submit AGS disks containing the GI and LT data in AGS format. Currently, about 200,000 GI data in AGS format have been stored in a centralized computer kept in the GIU and can be retrieved from the GIU through GEO's Geotechnical Information Infrastructure (GInfo) portal. The existing AGS data are generally in order and can be used for generating ground models. However, due to various technical reasons, usage of the AGS data appeared to be limited in the past. Recently, there has been an increasing demand for subsurface data in digital format for carrying out engineering analysis and 3D-applications such as Building Information Model (BIM). To facilitate these projects, it is considered necessary to make some modifications to the existing data format to enhance its user-friendliness and compatibility with commercial 3D modeling software. In March 2018 the UK AGS announced that they would no longer support the existing AGS version 3. As such, there is a need to review the specifications of the existing AGS version 3 in the GI and LT term contracts, and to upgrade the specifications to version 4. A study to upgrade AGS version 3 to version 4 by GEO is currently underway. The GEO is also taking this opportunity to review and make the GI and LT data more user-friendly and compatible with other geographical information system (GIS) and 3D modeling software.

This paper describes the major changes from AGS version 3 to version 4, and the progress of the study in the upgrading the AGS format with emphasis on the proposed enhancement of the AGS data in order to encourage wider usage of the AGS data in engineering projects.

1 INTRODUCTION

Subsurface data including ground investigation (GI) and soil and rock laboratory (LT) data, are used in most engineering projects especially those related to infrastructure development and geotechnical studies. For Government projects, ground investigation and soil and rock laboratory testing are carried out by Development Bureau's List of Approved Suppliers of Materials and Specialist Contractors for Public Works in Ground Investigation Field Work Category and Soil and Rock Testing Category respectively. In addition to the traditional GI and LT reports in hard-copy form, such contractors are required to submit digital GI and LT data by means of AGS disks. AGS data format is published by the Association of

Geotechnical and Geo-environmental Specialists (AGS). The requirement for preparing the AGS disks was introduced to GI and LT term contracts administered by Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) in 1993. Most Government GI contracts have also adopted similar requirements for AGS data. The current specification in Hong Kong is based on AGS version 3 with some modifications. The modifications are mainly based on recommendations from AGS (HK) which include a number of user-defined fields and custom pick lists to suit the HK practice. A checking program is used to ensure that the submitted AGS data meet the required format. In March 2018, the AGS UK announced that they no longer support the AGS version 3 as they immigrated to AGS version 4 (AGS4, 2017). In view of this, the GEO considered necessary to review the AGS version 4 format with the aim to incorporate and update the digital data format requirements in future GI and LT term contracts. As the Building Information Modeling (BIM) and other 3-D applications are becoming more important to all engineering projects, the GEO also take this opportunity to modify the AGS data format to make it more user-friendly and compatible with the 3-D modeling software.

2 BACKGROUND

The AGS data for Government projects are kept in the Geotechnical Information Unit (GIU) of the Civil Engineering Library (CEL) of the (CEDD). The GIU, which was established in 1983, forms part of the Civil Engineering Library (CEL) and houses a comprehensive collection of geotechnical data throughout the Hong Kong SAR. Since 1993, about 200,000 sets of GI Data have been stored in different types of storage media and kept in the Geotechnical Information Unit (GIU) (Plate 1). All AGS data have been transferred to a central server for better data storage and management. To enhance the efficiency of data retrieval, Digital Geotechnical Information Unit (DGIU) which is a system for retrieving the electronic copies of the GI and LT reports in the GIU was launched in July 2009. Such electronic data can be accessed in CEL or through Geotechnical Information Infrastructure (GInfo) (Plate 2).



Plate 1: AGS in Different Types of Storage Media



Plate 2: GI and LT Data Retrieved from GInfo

When the requirement for AGS was first introduced, the contractors were not entirely familiar with the AGS format and some contractors prepared the reports and AGS disks separately. By late 90s, such problems had largely been resolved. The existing AGS data are generally in order and can be used for generating ground models. However, due to various technical reasons, usage of the AGS data seemed to be limited in the past. Recently, there has been an increasing demand for subsurface data in digital format for carrying out engineering analysis and 3D-applications such as Building Information Model (BIM). Usage of the AGS data has significantly increased.

3 AREAS FOR ENHANCEMENTS TO EXISTING AGS DATA FORMAT

Although the existing AGS data are generally in order, some inconsistencies are occasionally found and enhancement works are required. These areas are: group name and order; heading name; unit in data values and data in critical fields. In Hong Kong, borehole logs and AGS data files are commonly generated from a single database using the gINT software package. The data in gINT database is then exported to the corresponding AGS data file. Since the AGS data files are prepared by different GI contractors, the above variations in the content among the AGS files from different contractors are inevitable.

For GI and LT which are carried out by different contractors, identity of GI Stations in GI fieldwork reports and laboratory testing reports may be different. An inconsistent borehole identity may result in incorrect ground information presented by a software package using the AGS data. To enhance the integrity and consistency, it is necessary to provide a linkage between GI and LT.

4 MAJOR CHANGES FROM AGS VERSION 3 TO VERSION 4

The changes in AGS version 4 can be broadly sub-divided into 1) Changes to Rules and 2) Changes to Groups. The rules are fundamental instructions to the production of AGS data file. The 'Changes to Rules' will change the data structure of the AGS data file. The groups collate the details, raw data and remarks of a specific type of information, such as project information, drilling information, geological information, field testing data and laboratory testing data. The 'Changes to Groups' will affect the locations of data to be collated and the content of information in the AGS data file.

4.1 Changes to Rules

The 23 Rules in AGS version 3 have been reduced to 20 rules in AGS version 4. There are 10 rules are removed from AGS version 3 (Table 1) and 7 new rules in AGS version 4 (Table 2), and 5 existing rules have also been modified in AGS version 4 (Table 3). These changes improve the consistency of AGS data file, such as the introduction of REQUIRED headings to solve the problem of missing data in some critical fields and give more control in the order of headings inside a group. These changes also enhance the presentation of data in the AGS data file by introducing data descriptor, removing the prefix and relaxing the limitations on the length of data and number of headings be contained in the AGS data file. The above changes facilitate the standardization of data presentation, such as the unit and type of data to be contained in DATA row. New functions such as record link and file transmission data under TRAN group also help users to trace the information between groups and between different revisions of AGS data files.

Table 1: Deleted Rules in AGS Version 3

Rule No.	Details
Rules 10 & 11	The asterisk(s) prefix (**/*) before GROUPs and HEADINGS is not required in AGS version 4.
Rule 12 to Rule 14	There is no limitation on character length of HEADINGS and data in AGS version 4.
Rule 16	The GROUP name cannot be repeated in AGS version 4.
Rule 17	There is no limitation on the number of HEADING inside each group in AGS version 4.
Rules 22 & 23	The prefix (**?/*?) before user-defined GROUPs and HEADINGS is not required in AGS version 4.
Rule 25	Listing the code used in chemical test under CODE group is not required. (CODE group is merged with ABBR group in AGS version 4).

Table 2: New Rules in AGS Version 4

Rule No.	Details
Rule 3 to Rule 6	These rules define new group item Data Descriptor and its usage. Each row shall be preceded by a corresponding Data Descriptor and the order of data descriptor is fixed as GROUP, HEADING, UNIT, TYPE and DATA.
Rules 8 & 17	These rules define new data field TYPE and its usage. The format of different type of data (e.g. number, text, date/time, etc) is defined and standardised.
Rule 11	This rule defines new data type Record Link and its usage. It allows the cross-reference of data under different GROUPS in the AGS data file.
Rule 14	This rule requires every AGS data file to contain the TRAN group. TRAN group contains the information related to file transmission (e.g. date, data file producer) and data status (e.g. Draft, Revision 1, Final).
Rule 15	This rule requires every AGS data file to contain the UNIT group which defines the unit of data (e.g. m, yyyy-mm-dd, kPa) under the DATA row.

Table 3: Changes to Existing Rules

AGS Version 3	AGS Version 4	Changes
Rule 1	Rule 1	The use of extended ASCII is permitted.
Rule 4	Rule 7	The order of HEADING shall follow the AGS Format Data Dictionary.
Rule 7	Rule 8	The data VARIABLES shall follow the unit and type described in the UNIT and TYPE field.
Rule 8	Rule 5	The use of quote (") inside the data field is permitted.
Rule 6	Rule 10	The HEADINGS are redefined from two categories of KEY or COMMON to three categories of KEY, REQUIRED or OTHER.

4.2 Changes to Groups

The total number of GROUPS in AGS version 3 has increased from 73 to 124 in AGS version 4. The major changes of GROUPS to GI include the renaming of HOLE group to LOCA group in order to reflect the fact that not all GI stations are boreholes. For the groups of instrumentation, the MONP and MONR groups in AGS version 3 are renamed as MONG and MOND which record the details of installed instruments and monitoring readings respectively. The details of piezometer are also recorded in MONG and MOND groups.

For the rock and soil laboratory testing, the ROCK group in AGS version 3 which contains data of all rock tests is no longer used. In AGS version 4, 8 rock test GROUPS are introduced to cover rock tests such as point load test, rock porosity and density test, unconfined compressive strength (UCS) etc. The soil test groups in AGS version 4 have been expanded to 36 groups which cover all soil tests listed in GEOSPEC 3.

There are new groups in AGS version 4. ERES group contains the data of environmental testing and GCHM group contains the data of geotechnical chemistry testing. New group DOBS has been added to record drilling observation whereas WADD to record water added during drilling.

5 MODIFICATIONS TO AGS VERSION 4 FOR PRACTICE IN HONG KONG

The current specification in Hong Kong is based on AGS version 3 with some modifications to suit the practice in Hong Kong. Some of these modifications are considered to be still valid for the current practice in Hong Kong and will be retained in AGS version 4 for Hong Kong. They include 1) a user-defined group ****?LEGD**, which is used to transfer the legend pattern code for different geological units, and 2) the pick list of legend code for the field **GEOL_LEG**. However, the ***GEOL_GEOL** field under ****GEOL** group, which records the formations of the sample based on the geological map published by GEO, will no longer be used.

Some new user-defined fields are planned to be introduced in AGS version 4 in Hong Kong. A unique station ID for GEO GI term contracts and GEO laboratory testing term contracts can help to link up the

sample location with the GI station and avoids confusion when there are common station ID (e.g. BH1, TP1) under different phase of GI. It can also avoid the conflicting IDs of GI stations (e.g. more than one BH1) shown by computer software when several AGS data files from different projects are imported.

A standardised pick list is planned to be introduced in data file in AGS version 4 to record the basic soil type (e.g. CLAY) with the general classification (ALLUVIUM) by a code ALLUVIUM-CLAY for example, or to record the rock type (e.g. GRANITE) with the weathering grade (e.g. completely decomposed) by a code CD-GRANITE for example. It can minimise the effort put on extracting the soil/rock type from the free text of geological description in AGS data files and facilitate the geotechnical engineers and geologists to produce a geological profile by using computer software.

6 IMPLEMENTATION OF AGS DATA FORMAT VERSION 4

The GEO is in the process of upgrading AGS data format from 3 to 4 by engaging a service provider, Golder Associates (HK) Limited. The details of the modifications to AGS version 4 in Hong Kong is being prepared (Golder, 2019). The draft specifications of the AGS data format for Hong Kong will be circulated to GI and LT contractors, and trial runs will also be carried out before finalising the specifications. It is anticipated that the new version can be rolled out in about a year's time and full implementation will take place upon the success of the trial runs. The finalised specifications will be incorporated in the new GI and LT term contracts.

7 FUTURE WORKS ON SUBSURFACE DIGITAL DATA

Although some enhancement will be made to the existing AGS data format to make it more user-friendly and compatible with other 3D modelling software, there are several areas which are worth paying attention for future enhancement.

7.1 Smooth interface between existing AGS data (version 3) with version 4 data

The existing AGS subsurface data (i.e. GI and LT data) kept in the GIU is a source of big data. Many geotechnical and geological software packages which are now commercially available can handle large amount of data for generating 3D ground models effectively and efficiently. Although the version 3 data format is no longer supported by the AGS (UK), this version of the data structure remains stable and can still be adopted by most AGS-compatible software. However, with the addition of the new groups and headings in AGS version 4, there will be a need to review and improve the structure of the AGS version 3 files to ensure that there is no major interface problem between the archived AGS files and the new version of the AGS files when they are used concurrently.

7.2 Add-value to initial AGS data

The AGS version 4 has the potential to handle multiple versions of the AGS files and therefore add value to the initial subsurface AGS data. By integrating the original GI data with the latest data collected throughout the project life cycle, the ground models can be refined for better design and effective construction control.

7.3 Application of AGS data to 3D modeling

The AGS data collected since 1993 consists of ground investigation data as well as soil and rock testing data. Whilst GI data facilitate 3D modeling in engineering projects, the improved data structures of the AGS version 4 would facilitate better utilization of the laboratory testing data for design analysis. More advanced software should be developed to integrate geotechnical properties in 3D ground models for better visualization and optimizing design (Aurecon, 2018).

7.4 Creating a territory-wide AGS subsurface database

The revised/improved AGS subsurface data can be centralized and put on a common platform for sharing. This valuable subsurface information can be used for further research and development to create large scale, or even territory-wide, 3D geological models. This could be useful for planning of future ground investigation works and geotechnical design at an early stage of civil/tunneling projects.

8 CONCLUSIONS

The GEO has been collecting subsurface data in AGS format since 1993 and is going to upgrade the existing version 3 to version 4 in a year's time. In addition to upgrading the version format, the GEO has taken this opportunity to make provisions to enhance the quality of the data format, including abilities to make multiple versions of the AGS subsurface data throughout the engineering project cycles, introduction of a unique identifier allowing cross-referencing of GI and LT data for the same investigation station. A standardized pick list will also be included in the specification to simplify the extraction of soil/rock type and weathering grades from the free text of geological description. The new AGS version will be adopted upon successful trial runs. Some ideas of further research and development works of the AGS data were also introduced. It is expected the enhancement can encourage wider usage of the AGS data in engineering projects.

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The prevalence and detection of UXO from the wartime bombing of Hong Kong

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ABSTRACT

Unexploded Ordnance (UXO) are explosive weapons that have been deployed but not detonated, either by design or malfunction. Hong Kong saw considerable military action during World War II, and Allied air forces dropped more than 4,000 bombs targeting the Japanese occupation forces. Bombing accuracy was poor and as many as 30% of the bombs failed to detonate. Most remain unlocated but are probably buried beneath, or close to current and proposed infrastructure. Accidental UXO detonation is a high consequence, low probability event, with potentially devastating effects. This risk can be managed, however, and appropriate allowances should be made at the design stages of a project for assessing the UXO risk, detecting and clearing any UXO likely to be present. Modern geophysical survey techniques are part of the clearance process and are discussed in more detail.

1 INTRODUCTION

1.1 Unexploded Ordnance. Definition and sources in Hong Kong

Unexploded Ordnance (UXO) are explosive weapons such as bombs, grenades or artillery shells that have been prepared for use and then deployed but failed to detonate, either by design or malfunction. UXO also include stockpiles of unused ordnance that were left behind or dumped after an armed conflict and are no longer under any control.

Hong Kong saw considerable military action during World War II, beginning with the invasion by Japanese forces in December 1941. The 18-day battle that followed the initial invasion was dominated by mobile heavy artillery from both sides, and the Kowloon Peninsula and Hong Kong Island between the north shore and The Peak became littered with UXO.

UXO including mines, torpedoes and bombs were also deposited on the seabed around Hong Kong, either fired during action or dumped after the conflict. Although much of the dumped UXO was made safe before disposal, a significant proportion was not and remains potentially dangerous (CED, 2000).

The initial invasion was followed by more than three years of Japanese occupation. Between 1942 and 1945 a heavy bombing campaign by Allied (mainly United States) air forces took place against the Japanese occupation forces, targeting dockyards and the former British military base at Kai Tak.

This paper focuses on the assessment and detection of air-dropped UXO resulting from the Allied

bombing campaign over Hong Kong, and does not cover UXO stockpiles, marine UXO or UXO from the initial 18-day heavy artillery battle.

2 AIR-DROPPED ORDNANCE. HONG KONG 1941 - 1945

2.1 Sources and types of air-dropped ordnance

Air-dropped UXO results from aerial bombardment of shipping, coastal defences, and military facilities. These are likely to be armed and will present a UXO risk. Bombs jettisoned by aircrew on the way to or from a target, which may or may not have been armed upon release, are another source.

Between 1942 and 1945 there were at least 51 days of bombing raids on the Kowloon Peninsula, Stonecutters Island, Kai Tak, the north shore of Hong Kong Island and ships in Victoria Harbour (Bailey, 2019). More than 4,000 bombs were dropped, with up to 30% failing to detonate (Heaver, 2017), and around 40% missing their target (Lee, 2014). The majority were AN-M64 (500 lb) and AN-M65 (1,000 lb) General Purpose bombs (Plate 2), with the AN-M64 being more common. Aerial photographs taken during the combat missions confirm the haphazard nature of the bombing. As many as 1,200 unexploded bombs therefore remain unaccounted for and are probably embedded in soft ground beneath or close to existing and proposed infrastructure, with some being discovered from time to time during construction activity, for example the two 1,000 lb (450 kg) bombs found at a Wan Chai North construction site in January 2018 (Plate 1). Military records are not comprehensive, however Table 1 provides an estimate based on available historical sources.

2.2 Accuracy of bombing

Under perfect conditions, the best World War II bomber pilots could drop their ordnance within 250 metres of their target (Hastings, 2010). However, the bomber pilots over Hong Kong were facing enemy fire and often inclement weather, and bombs often missed their targets or were jettisoned during the fighting. One such incident occurred on 24 August 1943 when 42 AN-M64 (500 lb) bombs all missed their targets in Kowloon and fell into the sea (Bailey, 2019).



Plate 1: One of two 1,000 lb bombs uncovered at a construction site in Wan Chai North in January 2018. (South China Morning Post, 2018)

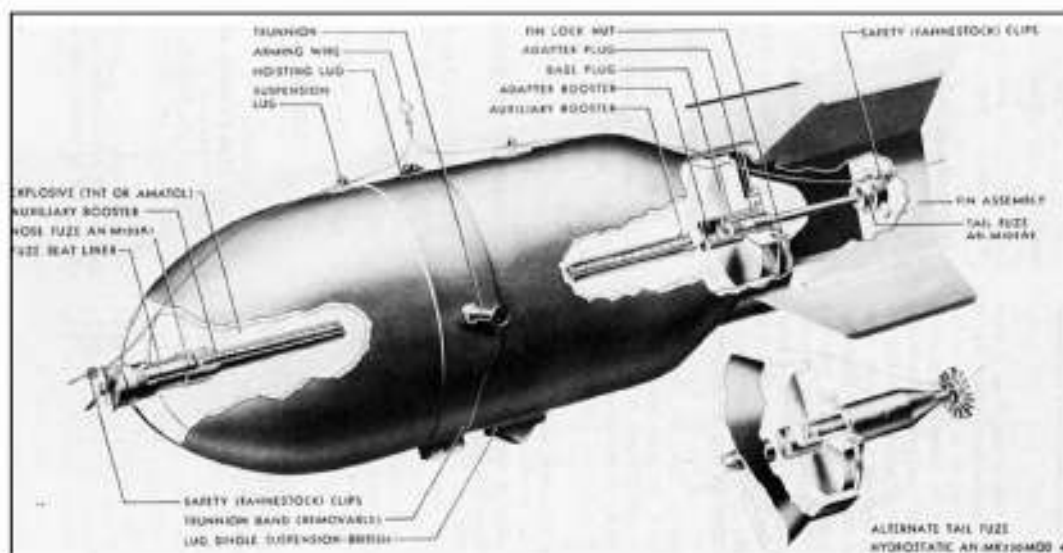


Plate 2: The ANM65 1,000 lb General Purpose bomb and the smaller 500 lb ANM-64 were the most common ordnance used by Allied forces during the aerial bombardment campaign. (Zetica, 2016)

Table 1: General purpose aerial bombs dropped on Hong Kong 1942 – 1945

U.S. Nomenclature	Weight (lb/kg)	Explosive Weight (lb/kg)	Dimensions (mm)	Estimated total quantity deployed*	Demolition targets for this category of Ordnance
AN-M30	100 / 45	54 / 24	1021 x 207	400	Railroad equipment, tracks, small buildings, ammunition dumps, planes on ground
AN-M57	250 / 114	123 / 56	1153 x 27	100	As above plus terminals, ammunition dumps, destroyers, submarines
AN-M64	500 / 227	262 / 119	1443 x 361	3,200	Steel railroad bridges, subways, concrete docks, light cruisers
AN-M65	1000 / 454	530 / 241	1704 x 478	360	Reinforced concrete bridges, steel bridges, piers, approach spans, medium cruisers
AN-M66	2000 / 910	1061 / 482	1765 x 530	12	Massive reinforced concrete and suspension bridges, heavy cruisers, battleships, dams

*Estimate based on U.S. air bombing records, and Bailey 2019.

The most intense bombing was delivered by the U.S. Third Fleet on 16 January 1945 (Plate 3), when more than 900 bombs ranging up to 2,000 lbs were dropped on Kai Tak, Kowloon and Taikoo docks, and ships in the harbour (Kwong & Tsoi, 2014). Many of these may still be embedded somewhere in the foreshore or below subsequent reclamation.

3 THE SPREAD AND BURIAL DEPTH OF UXO

3.1 Geological Setting

The major bombardment locations included the Kowloon Peninsula, Lai Chi Kok, Kai Tak and northern Hong Kong Island. These areas are covered with recent deposits interrupted by older Mesozoic igneous rock (Fyfe et al, 2000, Kirk et al., 2000), as described in Table 2 below.



Plate 3: (left) Bombing raid on 16 January 1945; in the background are Queens Road West, Sheung Wan and Mid-Levels (Wikimedia, 2016). (right) 2018 infrastructure superimposed over the same locations

Table 2: The geological setting of major bombardment locations

Location	Description
Hong Kong Island (north shore)	Reclamation materials bounded by Mount Davis Formation, Kowloon Granite and Mount Butler Granite to the south.
Kai Tak	The surface geology of the area mainly includes reclamation materials, alluvium and colluvium. Kowloon Granite and Mount Butler Granite are found at the north and east respectively.
Kowloon Peninsula	The shore is covered with reclamation material while Kowloon Granite dominates the central portion with local superficial Quaternary deposits of alluvium and colluvium.
Lai Chi Kok	Reclamation materials covered most of the area with Kowloon Granite and Mount Davis Formation found at the south near the shoreline. To the north, Sha Tin Granite, Kowloon Granite, Needle Hill Granite and superficial Quaternary deposits of alluvium and colluvium are found.

3.2 Reclamation before and after World War II

Reclamation before and after World War II is shown in Figure 1. This affects the geology on both sides of Victoria Harbour and coincides with many bombardment targets including the Hong Kong and Whampoa Dockyards as well as the former Kai Tak Airport. The significance of the reclamation timing is that UXO may be sitting on top of older reclamations, then covered by subsequent reclamation. Burial depth is therefore complicated by reclamation history, and the type of fill used. Marine sand was often used as fill material during reclamation in Hong Kong, either dredged from Victoria Harbour (Guilford, 1997 and Henry et al., 1961) or further afield. Dredged sand is also a potential source of UXO contamination.

3.3 Natural geology

On both sides of the harbour, the seabed is covered by a layer of fill (original seabed materials having been removed by dredging). Further offshore, sand and muddy sand are found around the western, central and eastern parts of Hong Kong island whilst around 15m of alluvial clay, silt and sand of the Pleistocene Chek Lap Kok Formation resting on an irregular surface of weathered rock is found between the shoreline of the Kowloon Peninsula and Hong Kong island. The remaining seabed areas are covered with Holocene marine mud and sandy mud of the Hang Hau Formation which generally rests on the Chek Lap Kok Formation. (Strange & Shaw, 1986)

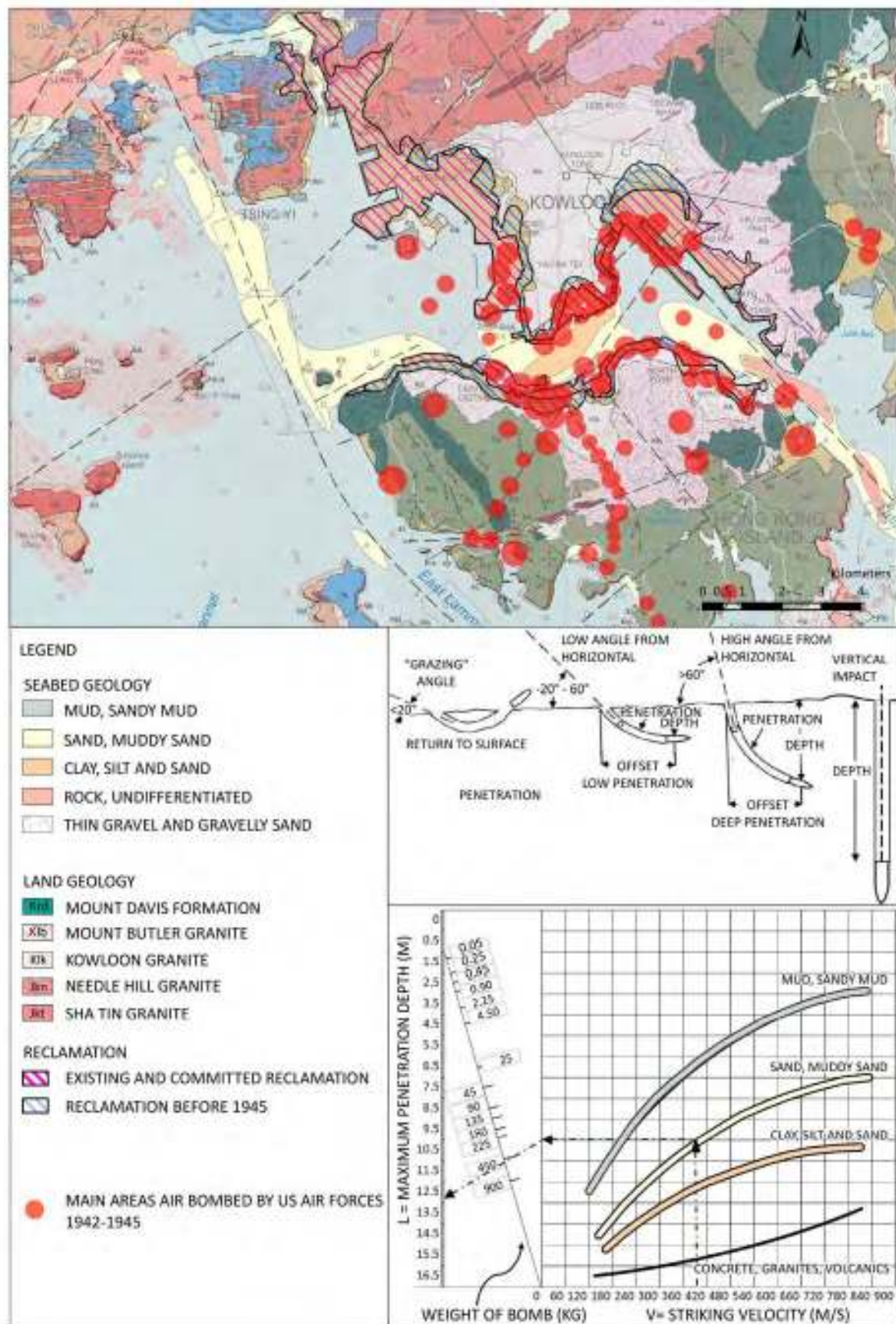


Figure 1: Surface geology, reclamation phases, aerial bombing intensity and UXO penetration depth (after Liu 2018b, Simms et al 2004; Fyfe et al, 2000; Kirk et al, 2000)

3.4 Recent UXO discoveries in Hong Kong

UXO discoveries are underreported (especially in the construction sector) but average around ten per year. Some of these discoveries coincide with the target bombardment locations, but there is a wide margin of error (Table 3). Many of the larger finds have been kilometres away from any known objective.

Table 3: Significant recent UXO discoveries in Hong Kong

Year	Location	Qty of UXO	Comment	Source
1993	Seabed near Tsing Yi	1	500 lb bomb, dredger HAM308, declared a total loss	Liu, 2018a
2013-18	Pok Fu Lam, Stanley, Ma On Shan	35	various	Liu, 2018a
2000-02	Construction site Penny's Bay, Lantau	~200	Over 400 UXO (mainly shells, mines, bullets) dredged from East Lamma Channel.	Heaver, 2017
2006	Tonkin Street and Fuk Wing Street, Kowloon	588	Munitions cache. Water Services Department excavation	Heaver, 2017
2013	Tai Tam Country Park	1	2,000 lb bomb	Lee, 2014
2014	Hau Tak Lane, near the racecourse	1	2,000 lb General Purpose bomb	Heaver, 2017
2014	North Point	1	36 kg bomb, Japanese	Gallagher, 2014
2017	Pok Fu Lam construction site	1	500 lb bombs	Heaver, 2017
2018	Harbour Road and Tonnochy Road in Wan Chai	2	1,000 lb bombs	Liu, 2018a

3.5 UXO burial depth

The maximum burial depth of any UXO may be estimated. This is an early and important phase of risk assessment. A bomb falling into water loses velocity very rapidly and burial in the seabed is minimal (Chu et al 2010). However, on land the burial depth of any UXO is a function of geology (geotechnical properties), the weight of the ordnance, the striking velocity, impact angle and nose profile. The penetration depth chart in Figure 1 has been prepared using the nose profile common to AN-M64 and AN-M65 bombs, and the ground properties common to the study area. An example for a 1,000 lb (450 kg) AN-M64 bomb landing at 420 m/s is provided, estimating a penetration distance slightly less than 13m. However, penetration distance may not equal burial depth due to the 'J-curve effect' (Figure 1) caused by the impact angle.

3.6 UXO risk assessment

Experience shows that the risk of casualties from inadvertent UXO detonation has been very low. However, this is an unpredictable, high consequence and low probability event (Stone et al, 2009). The UXO are unstable and still contain large amounts of active explosives, and may only need vibration, impact, motion, a magnetic field or an electrical current to initiate the detonation process. UXO may be buried for years in land or the sea floor and then reappear – sometimes repeatedly, due to geological processes and human activity. Examples of inadvertent detonation on construction projects are provided in Table 4.

Table 4: Inadvertent detonation of World War II UXO on construction sites resulting in injuries or fatalities

Year	Location	Comment	Source
26 April 1978	Münsterland, Germany	Excavator fitted with drilling tool encountered UXO that immediately exploded. Three fatalities.	Oord, 2018
24 Feb 1993	Yau Ma Tei, Hong Kong	Incendiary bomb detonated while digging on construction site. 700 persons evacuated.	South China Morning Post, 1993
25 Feb 1993	Tsing Yi waters, Hong Kong	500 lb bomb, dredger HAM308 declared a total loss. One injury.	Liu, 2018a
23 Oct 2006	Aschaffenburg, Germany	Motorway construction works. Aircraft bomb detonated. One fatality, one injured and site vehicle destroyed.	Spiegel, 2006
3 January 2014	Euskirchen, Germany	Excavator struck bomb (likely to be 1,000 lb) One fatality and eight wounded (two critical)	BBC, 2014
11 October 2017	Kuala Lumpur, Malaysia	One fatality, two critically injured (loss of limbs) after bomb exploded at subway construction site	The Straits Times, 2017

The main effects of UXO detonation are shock, blast, heat, shrapnel damage and noise (RATCH, 2013), occurring very rapidly and resulting in physical destruction and effects on the environment. Consequences of UXO detonation are dependent on several factors, including burial depth and type of UXO. In extreme cases, multiple fatalities to personnel and severe damage to construction equipment may result. In order to quantify and assess the risk of accidental UXO detonation on site, a risk assessment matrix (Figure 2) and risk management framework (Figure 3) may be used. The matrix assesses UXO risk based on three major elements: probability of encountering and detonating UXO, detonation consequences, and associated severity.

3.7 UXO environmental contamination

Detonation or corrosion may also release environmental contaminants, including metals such as copper or zinc from the bomb casing, lead or mercury from the detonator, organic aromatics (TNT, TNB, RDX and related products) from the explosive charge, as well as white phosphorus or other chemicals from incendiary devices (DJIM 2015). This can cause environmental damage including direct toxic effects on organisms, subsequent bioaccumulation in the food chain, and effects on water resources (BLMP 2011).

		Likelihood of Risk Event (%)				
		A	B	C	D	E
		Rare (< 1%)	Unlikely (1% – 10%)	Possible (10% – 25%)	Likely (25% – 50%)	Almost Certain (> 50%)
Severity / Consequence Class	1	LOW 1	LOW 2	LOW 3	LOW 4	MEDIUM 5
	2	LOW 2	LOW 4	LOW 6	MEDIUM 8	MEDIUM 10
	3	LOW 3	MEDIUM 6	MEDIUM 9	MEDIUM 12	MEDIUM 15
	4	MEDIUM 4	MEDIUM 8	HIGH 12	HIGH 16	HIGH 20
	5	HIGH 5	HIGH 10	HIGH 15	HIGH 20	HIGH 25

Figure 2: Risk assessment matrix (Berg, 2018)

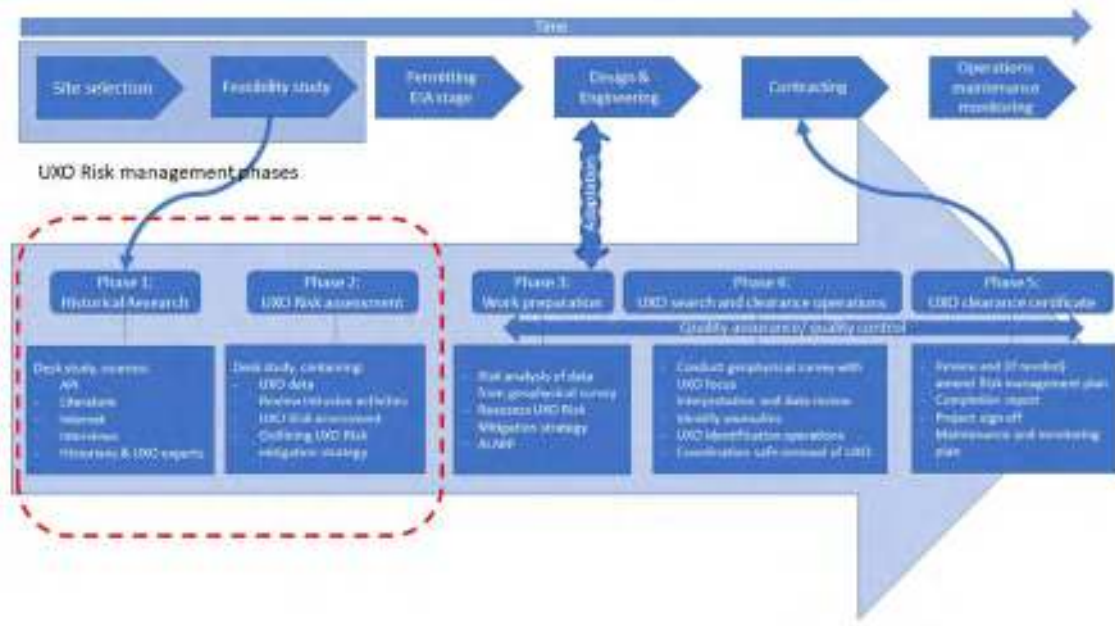


Figure 3: UXO risk management phases (after Berg, 2018)

3.8 Unintended detonation of UXO

Unintended UXO detonation may be caused by impact or crushing by heavy equipment, friction, static electrical charge, heat, or sympathetic detonation caused by the detonation of an adjacent UXO. Typical activities that can cause inadvertent detonation during construction works include manual or machine excavation, geotechnical drilling, jetting, probing or mixing, deployment of jack-up barge legs, anchor deployment, dredging and piling.

4 UXO SURVEYS

4.1 General approach. Marine and land

To minimise the risk of unintended detonations of UXO due to intrusive activities such as drilling or piling, UXO surveys are executed beforehand. A typical UXO survey comprises of three stages: a desk study to identify the areas of bombardment from historical records and aerial photos; shallow surface detection of UXO in the identified areas using ground penetrating radar; and finally a geophysical survey using magnetometer techniques. The target of magnetometer surveys is the interfering magnetic field caused by ferrous material in the shallow surface. This helps determining the target locations of invasive investigations in the shallow subsurface and provides rapid coverage of large areas.

4.2 Desk Study – Aerial photo interpretation and analysis of historical records

The desk study forming the initial part of a UXO survey typically comprises the analysis of relevant aerial photos and historical records, geo-referencing and interpreting the aerial photos to identify and map bombardment-related features, followed by the assessment and identification of areas with UXO-associated risks (Meixner & Eckstein, 2016)

Aerial photo interpretation is used to aid the identification of the bombardment locations. Evidence of bombardment such as destruction of facilities and bomb craters may be distinguished by comparing aerial photographs taken at different times. For instance, in Figure 1 the smoke from the bombarded areas can be clearly seen. Although reliable identification of potential UXO sites can be relatively complicated due to the need for high image quality, a risk zone of about 50m around visible bombardment locations has proved to be a reasonable radius for the likelihood of encountering UXO. (Meixner & Eckstein, 2016)

Historical records (military and non-military) can provide information on bombardment locations, types of ordnance, and reclamation phases of the study area. This can provide additional evidence to

confirm the photographic interpretation and assist in the estimation of approximate locations and depths where UXO might be discovered, allowing selection of the most appropriate detection equipment.

4.3 UXO search and clearance operations

Search and clearance operations are an integral part of the risk mitigation strategy. Following the desk study, surface detection and discrete intrusive detection surveys may be performed. Detection methods include magnetic, electromagnetic and ground penetrating radar. Magnetometer methods are commonly used for intrusive surveys that penetrate the ground at discrete locations such as proposed pile locations (Fugro, 2019).

4.4 Magnetometer techniques

Magnetometer surveys may be used for land and offshore/shallow water projects. They are a passive, low cost and fast detection method. (Mu et al, 2018). There are variations with the data processing techniques, however the principle is detection of ferrous objects by their magnetic field. They can be extremely sensitive and are routinely capable of detecting larger (>250 kg) UXO within six meters of the probe position (Eriksen et al, 2016). These surface magnetometer/gradiometer surveys provides immediate results in the form of target plots which form the basis of explosive ordnance clearance (Fugro, 2017). They provide rapid coverage of large areas but provide limited information on detection depth/penetration. Therefore, in addition intrusive methods are usually used, such as magnetometer cone penetration test or drillhole magnetometer testing.

In high risk areas with soft geology, UXO may be buried at much greater depths than surface surveys can detect, and the magnetometer may be fitted to a Cone Penetration Test (CPT) rig to provide clearance at discrete at-depth locations such as proposed pile locations. A typical proprietary system is Fugro's MagCone® (Plate 4), which combines a gradient magnetometer with a standard 44mm CPT cone and allows the detection of magnetic anomalies such as UXO while simultaneously recording geotechnical parameters. This allows intrusive UXO survey in soft soil more than 25m deep with a detection radius up to 2m from the cone. The system can be used both on land or over water.

For deep detection in areas unsuitable for CPT, such as dense gravel or made ground containing rubble, a magnetometer system may be used with a drilling tool. The magnetometer system is lowered to the bottom of the drillhole at regular intervals and confirms the absence of any UXO within range of the tool. The system is then removed, and drilling continues within the cleared depth before the magnetometer is lowered again, to clear further drilling.

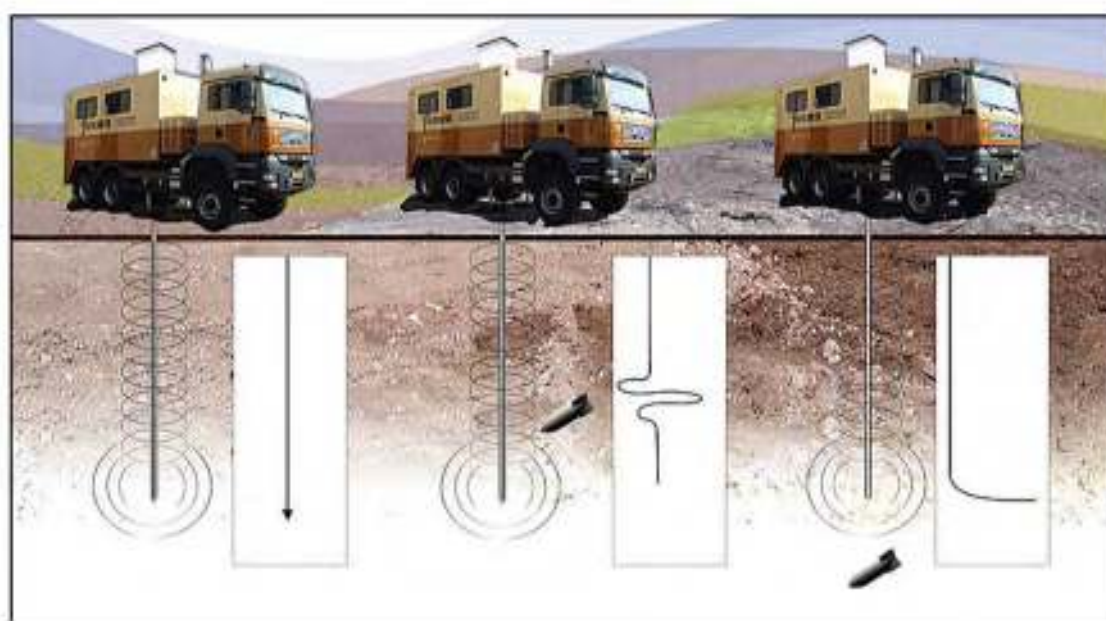


Plate 4: CPT magnetometer survey to detect UXO in soft ground (Fugro, 2017)

4.5 Challenges with UXO survey

The major challenge for UXO surveys is the misidentification of magnetic anomalies. The detection of UXO below the subsurface requires a measurable contrast between the target and the surrounding environment. Therefore, the effectiveness of geophysical methods is impacted by the presence of other metallic objects such as utilities or structures above or below the ground surface affecting the sensitive high resolution equipment needed for UXO detection, although combining magnetometer and gradiometer data can help (Fugro, 2019). However, experienced operators are always needed for the proper verification and interpretation of the data and real time data acquisition.

CONCLUSIONS

As many as 1,200 unexploded bombs may remain beneath or close to Hong Kong's existing and proposed infrastructure. Uncontrolled UXO detonation is a low probability event, with potentially catastrophic consequences. Previous incidents have resulted in damage or loss of infrastructure, serious injury and loss of life. This risk can be managed, and appropriate allowances should be made at the design stages of a project for risk assessment and management. The risk management process may include geophysical survey techniques performed over the ground surface or water for shallow detection, or at depth using a CPT or down-hole magnetometer tools.

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The Use of Floating Breakwater Technology in Hong Kong

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ABSTRACT

Reclamation in Hong Kong is continuously becoming more complex, is larger in scale, and is exposed to higher risks and consequences. The demand for innovative and creative marine solutions to optimize the performance of such complex works is also increasing. With a view to overcome the constraints and issues associated with inclement weather on the Works, such solutions are required to plan for difficult marine access, to ease installations, to minimize susceptibility of site damage, to maintain consistency and reliability and to improve programme certainty.

A floating breakwater is a temporary protection system with the sole purpose of reducing wave height. The merit of the floating type is that the breakwater does not require a foundation and is easily maneuverable by marine methods. The flexible and modular breakwater units are prefabricated such that it is easy-to-install with the manageable size, can be connected over long distances and requires minimal maintenance. This paper discusses Hong Kong's application of floating breakwater technology that eases some challenges in today's reclamation works; and ultimately enables engineers and scientists to enhance complacency with the quality and reliability of temporary wave attenuating systems.

The first 250m long temporary floating breakwater was installed in Tseung Kwan O, Hong Kong in early 2018 to protect the reclamation site of the Contract "Tseung Kwan O – Lam Tin Tunnel – Road P2 and Associated Works". The floating breakwater provides a safe and comfortable working marine environment during construction, and has successfully enhanced the resilience of the project after being subject to typhoons in 2018.

1 INTRODUCTION

The reclamation of land from the sea has been ongoing in Hong Kong due to the dire need of land supply for the public and the increase in demand for infrastructure developments has recently triggered an outburst of reclamation activities. Many of the successful techniques used today are existing methodologies adopted around the world including the Netherlands, South Korea, Singapore, Japan, USA and Bahrain.

Construction challenges faced include providing marine access of vessels, maintaining a safe and comfortable working environment, meeting the tight construction programme, fulfilling of environmental protection requirements and minimizing weather-related damage in the site. For the latter challenge, measures implemented during daily operations and typhoons events require a collaborative approach

from all parties which stimulate engineering, administrative and protective measures in addition to fundamental safe practices.

Floating breakwater technology has been widely developing since the 1960s with the purpose to reduce or attenuate wave height and thus reducing wave loads on structures, vessels and equipment. A temporary floating breakwater is a protective measure that is fast to construct, has a small footprint, is foundation free, is economical and environmentally safe to tackle inclement weather conditions. This paper discusses our proactive approach using the first 250m long temporary floating breakwater to protect the construction site in Tseung Kwan O, Hong Kong from waves during the temporary reclamation stage.

2 FLOATING BREAKWATER TECHNOLOGY

2.1 Overview of Floating Breakwaters

The main purpose of a breakwater is to protect near or on-shore facilities, such as structures, harbours and moored vessels from loads due to wave actions. Most breakwater systems are passive, which are fixed or floating, and attenuate waves by reflection, dissipation, transmission or a combination of these mechanisms. The major types of breakwaters consist of old and well-established fixed structures, such as rubble mound, precast concrete units and steel sheet pile cofferdams; Transportable structures, such as caisson type units and sunk-in-place barges; Tethered structures, such as buoyant floats; And free-floating structures, such as pontoons, flexible membranes and tire modules (Refer to Plate 1).

With the requirement of lesser degree of protection compared with permanently fixed breakwaters, it is encouraging to use temporary breakwater systems. The floating breakwater provides wave protection with the advantages of mobility, low cost, and fast construction. The documented concept of floating breakwaters has a rich history. Early records of non-bottom-fixed breakwaters dates back to 1811, pioneered by General Bentham, the Civil Architect and Principal Surveyor of the Royal Navy of Great Britain. In 1905, J. Joly proposed floating breakwater concepts as illustrated in Figure 1 to the Royal Dublin, Society for the harbours of Arklow and Wicklaw on the east coast of Ireland. Out of necessity, the extensive use of floating wave barriers was later applied during World War II where portable barge-type units ensured safe offloading of soldiers and materials.

Today, many existing harbours and marinas around the world have floating breakwaters such as floating pontoons to protect small moored crafts and working platforms from short period waves and vessel wake. Its applications are proven to be effective for partial protection at nearshore zones, on exposed coastlines, at sheltered bays and within estuary locations with less severe climates. Current development is continuously evolving floating breakwaters with better performance for offshore marine environments. An example of floating breakwater currently being developed (Chan, et al, 2015) is indicated in Plate 2 consisting of two cylinders with a submerged flexible mesh cage containing suspended rubber hollow balls which absorb wave energy.

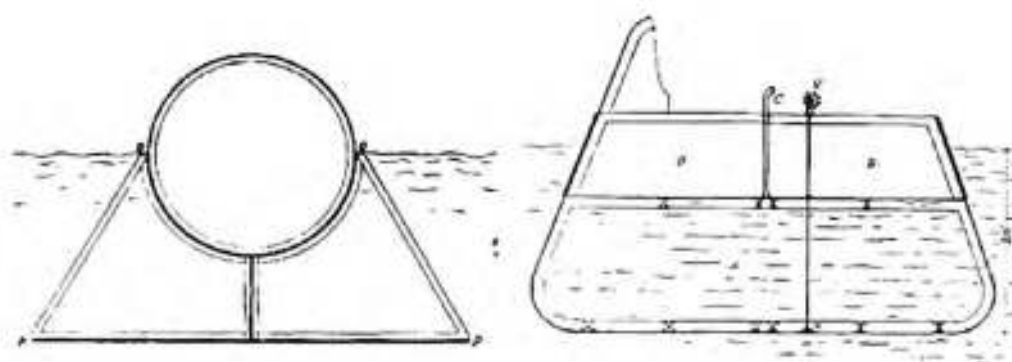


Figure 1: Early illustrations of floating breakwater concepts (Joly, 1905).



Plate 1: Photograph of recycled tire floating breakwater (Topper Industries Inc.)



Plate 2: Current floating breakwater technology (Chan, et al, 2015)

In Hong Kong, berthed floating barges are commonly used to temporarily calm waves during Dragon Boat races. Use of floating breakwaters in the Hong Kong environment for construction has its merits due to the thick and low strength of Marine Deposit below the seabed, high water draught, sensitivity to water quality impact and where potential reclamation sites are in semi-sheltered areas.

2.2 Existing Studies on Floating Breakwater Performance

Many published studies have found good performance with different types of floating breakwater systems. However, the overall acceptance of the floating breakwater concept remains debated due to the practitioners confidence of the wave attenuation effectiveness. A wide range of proprietary floating breakwaters is available in today's market, each with individual benefits. Detailed study of floating breakwater effectiveness is beyond the scope of this paper and any detailed comparison of design types warrant little economic merits for the Project. Reference has been made to the specialist suppliers which can introduce breakwater innovation that achieve their wave attenuation intent.

The main load transfer mechanism of the floating breakwater is transmission, where the wave load is transmitted to the anchor seated on the seabed. Should the floating units be filled with a fluid or foam, the wave load is then partially dissipated by internal dampening effects. The design of floating breakwaters is generally grouped into three categories: simple analytical models, physical hydraulic

models and numerical models. Figure 2 indicates a sketch of a physical test setup of a scaled model floating breakwater by Ripken, 1960. Despite the complexity and myriad of conditional assumptions, the main design objective is to obtain a rough estimate in the transmission coefficient of the breakwater (wave height attenuation) and the required mooring force. The main environmental inputs are the wave height, length, steepness, direction, relative depth and water draught. Regardless of mass, inertia and geometry properties of different proprietary floating breakwaters, the general trends have been concluded from existing studies:

1. Wave attenuation ability decreases as water draught increases;
2. Wave attenuation ability decreases as wave height, length and steepness increases;
3. Wave attenuation ability decreases as breakwater length and width decreases;
4. Wave attenuation ability decreases as water depth below breakwater increases;
5. Wave attenuation ability decreases as relative depth of breakwater decreases;
6. Wave attenuation ability is optimal when the breakwater alignment is perpendicular to the incident wave;
7. Mooring anchor load increases as stiffness increases taking into account length and direction of anchor wire;
8. Mooring anchor load increases as wave height and length increases.

Literature has other influences of wave attenuation potential including the wave period, internal fluid and cavity volume, mass density of water, dynamic viscosity of water, velocity of water and gravitational acceleration.

The use of temporary floating breakwaters in Hong Kong is recently emerging in construction works and will become more widespread on upcoming projects involving reclamation, ground improvement, seawall modification, intake and outfall construction and associated marine works. As wave attenuation data in the local marine environment becomes available, an opportunity for further studies will be open for practitioners.

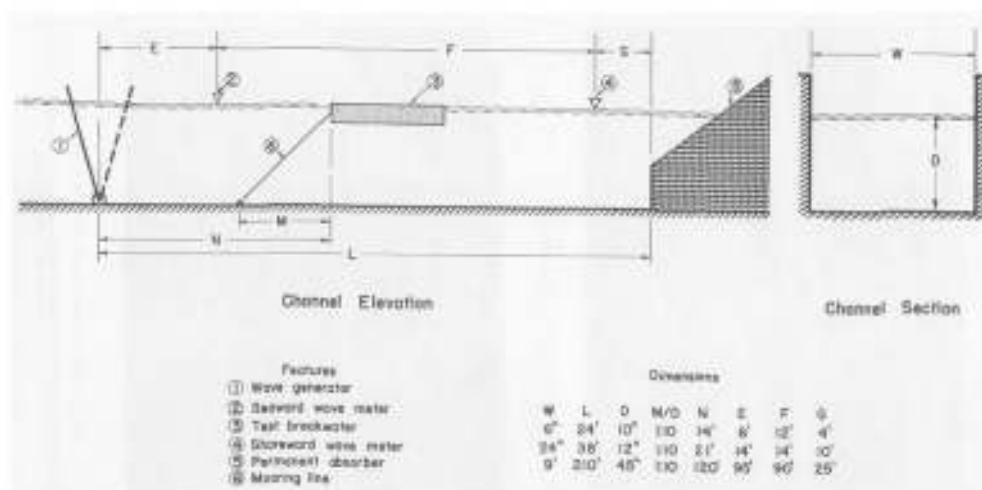


Figure 2: A floating breakwater physical test setup (Ripken, 1960).

3 THE SITE

3.1 Site Description

The first use of a temporary floating breakwater of 250m length in Hong Kong is in Tseung Kwan O, in early 2018 for the construction on the Contract "Tseung Kwan O – Lam Tin Tunnel - Road P2 and

Associated Works” (P2). The Contract commenced in July 2016 and includes reclamation of around 3 hectares along the northwest coast of Junk Bay to enable construction of a dual two-lane Road P2 of approximately 800m long to connect the Tseung Kwan O – Lam Tin Tunnel from the Tseung Kwan O Town Center. Figure 3 indicates a map to the P2 site location.



Figure 3: Site location map of Tseung Kwan O – Lam Tin Tunnel Road P2 Project.

The Employer, Civil Engineering and Development Department (East Development Office) (CEDD), awarded the Contract to CRBC-Build King Joint Venture (CRBC-BK JV) as the Contractor to carry out the Works. Meinhardt Infrastructure and Environment Ltd. (MIEL) is engaged as the Contractor’s Designer responsible for the design of temporary works and the Cost Savings Designs for permanent works.

The marine works under the contract include dredging marine deposit consisting of Soft Clay and Loose Sand, placement of geotextile, installation of prefabricated band drains, construction of the permanent seawall, filling of sand and reclamation fill, and installation of seawall coping and rock armour. The initial temporary works include deployment of a perimeter silt curtain and construction of a temporary steel sheet pile environmental cofferdam with a double water gate to confine the reclamation footprint. Marine transportation is deployed to deliver construction materials to and from the site.



Plate 3: Aerial photograph of the P2 Project site area on February 2019

The Project site is located in Junk bay, a semi-sheltered inlet of the Sai Kung District in the east of Hong Kong’s Victoria Harbour. The northern tip of the bay lies in Tseung Kwan O and to the west is the access road to the Junk Bay Chinese Permanent Cemeteries on the slopes of Chiu Keng Wan Shan

(照鏡環山). The estimated critical fetch length, which determines critical wave loads, is towards the south to Po Toi Island averages to 13km. The seabed level is generally -7mPD to -8mPD along the southern perimeter of the site. With reference to tidal data from the Hong Kong Observatory (HKO) monitoring locations at Quarry Bay and Waglan Island, the extreme seawater level with 2-year return period is +2.7mPD and the lowest seawater level is -0.3mPD. Subsequently, the water draught varies between 6.7m to 10.7m. Ground investigation indicates a stratigraphy consisting of approximately 4m thick Marine Deposit Sand and 7m thick Marine Deposit Soft Clay underlain by Alluvium of Firm to Stiff Clay on top of Completely Decomposed Tuff at approximately 16m depth.

3.2 Site Issue

Hong Kong's sub-tropical climate subjects the site to regular high wave loads and typhoons conditions with extreme winds and waves between the months of July and September. Typhoons which trigger the highest Typhoon Warning Signal No. 10 is infrequent with 1 in 70 typhoons reaching the ultimate signal according to HKO. Three number 10 Signals were issued after 2010, two of which had Typhoons recently touchdown within 100km of Hong Kong in 2017 and 2018. Super Typhoon *Hato* hit Hong Kong on 20 to 24 August 2017 and Super Typhoon *Mangkhut* hit on 7 to 17 September 2018, both of which occurred during the construction at P2. The latter typhoon recorded a maximum 60-minute mean wind speed at Waglan Island of 161 km/h, the second highest recorded speed at the station. A maximum seawater level of +3.73mPD was measured at the Quarry Bay tide monitoring station.

During construction, vessels, equipment, the temporary environmental cofferdam, the existing seawall and reclamation leading edge within the work area are regularly subject to adverse conditions from waves and vessel wake. Protective measures are routinely taken to avoid or minimize weather-related disturbance or damage during construction, particularly during the typhoon season.

Up to 10 number of vessels are engaged during construction which include non-self-propelled Flat Top Barges, Tug Boats, Derrick Lighters, Crane Barges, Pelican Barges, Worker Boats and Guard Vessels. All vessels refuge to the nearest typhoon shelter in the event of an evacuation from the site due to incoming typhoons.

3.5 Measures to Reduce Weather-Related Damage

The mutual aim of all parties is to ensure that the Works are carried out in a safe and comfortable working environment, and marine vessels and equipment are kept in good working condition during construction. In addition to reliance on marine workers adhering to safe working practice, the Employer, the Contractor, and the Designer have carried out a combination of administrative, engineering and protective measures to place the safety of workers and the public as paramount.

Administrative precautionary measures, include providing special safety training for all personnel working over water or adjacent to water, carrying out regular safety meetings, deploying guard boats to patrol, installing temporary lighted anchor buoys and signage, assigning marine restricted areas, preparing Marine Construction Safety Plan, Typhoon Evacuation Plans and Contingency Plans. The Marine Superintendent and boat skippers closely checks the weather forecast from HKO and suspends any marine operations if the typhoon signal no. 3 is hoisted; All vessel and barges are subsequently evacuated to typhoon shelters in To Kwa Wan, Kwun Tong and Yau Ma Tei. Preventative measures include keeping the work place tidy and clean, keeping safe stacking of materials, securing of doors and hatch covers, fixing loose parts and objects and providing personal protective equipment and lifesavers for workers. Engineering measures include constructing temporary bollards for construction berthing, employing real-time GPS systems to monitor vessel movement with collision alarm and constructing a temporary environmental cofferdam taking into account wave loads around the perimeter of the site. Despite our most determined efforts to protect the site, additional measures were deemed necessary to further isolate the P2 site from wave associated risks by an innovative breakwater system.

4 THE ADOPTED FLOATING BREAKWATER

Our genuine collaboration and innovative spirit had driven many brainstorming sessions with the common goal to find an additional wave protection measure for the P2 site. Different systems were considered including bottom-fixed rigid breakwater, inflatable dam, floating scrap tire breakwater and floating pontoons. The most feasible temporary breakwater options were narrowed down to construction of a concrete block seawall, to berth a line of vessels or to install a floating breakwater. The berthing of five floating vessels was not considered as the large footprint would encroach beyond the site boundary and would cost approximately fourteen times more than that of the floating breakwater. The concrete block seawall is unstable when bearing on the Soft Marine Deposit, and would cost approximately five times more than that of the floating breakwater. Also the concrete block method is discouraging as large volumes of construction and demolition material would be generated upon dismantling.

A modular floating breakwater was finally selected due its small footprint which remained within the work boundary, is relatively low cost, is quick and easy to install and handle, requires low maintenance and has good alignment flexibility. The floating breakwater does not rely on a fixed foundation and their associated marine installation works considering the low strength Marine Deposit foundation. The nuances of termination and tolerance requirements for marine piling are alleviated with a floating system. When compared with concrete blocks or floating pontoons, construction vessels within the vicinity are not prone to damage upon contact with the breakwater material. Of the many variety of floating breakwaters, the particular type was selected due to market availability, fast delivery time of stocked units from the nearby Mainland China factory and competitive pricing. The floating breakwater is located along the southern boundary of the site where sufficient space is available, and where most of the critical waves can be intercepted.

4.1 Details of the Floating Breakwater

The adopted floating breakwater consists of individual modular units made from hollow High Density Polyethelene (HDPE) shells. Each unit is light weight and suited for manual lifting. The assembled breakwater section consists of 6 nos. of units in a pyramid shape layout stacked to 3 units high (1.8m) and 5 units wide (2.8m) as indicated in Figure 4. The bright red and white colours make the floating breakwater easily visible to passing vessels in addition to the installed lighted marker buoys. Each unit consists of 4 knuckles and is connected by feeding through stainless steel cables so a 10m long segment of floating breakwater is fabricated. Each segment is secured by 4 nos. of chains, each anchored to concrete blocks seated on the seabed. Attaching additional unit layers or adjusting the anchor tightness easily adjusts the 1m freeboard of the breakwater. The openings between units allow water flow through the breakwater to avoid water stagnation.

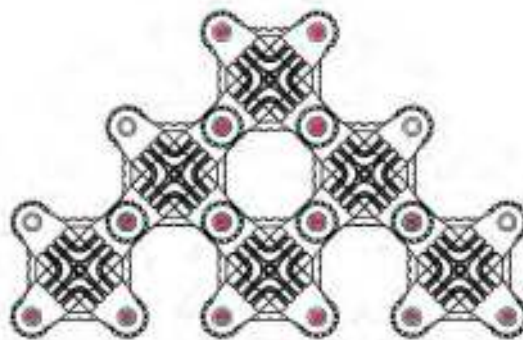


Figure 4: Schematic cross section of the adopted floating breakwater.

4.2 Installation Methodology

The setting out alignment of the floating breakwater is marked with stone sinkers tied to temporary marker buoys, which assist the vessel operators during breakwater segment placement. Whilst on land, the breakwater units are manually assembled to the final breakwater arrangement of 10m

long segments. The fabricated breakwater segments and the concrete blocks are then transported by Derrick Lighter to the designated location. A Flat Top Barge is deployed as a work platform where the breakwater segments are laid out, connected together and set out along the proposed alignment. The length of anchor chain is determined by a sum of the water depth at the high tidal level, the maximum wave height, the length of chain required to tie the concrete blocks and breakwater and the predicted settlement of concrete blocks. The Flat Top Barge steers forward while laying floating breakwater

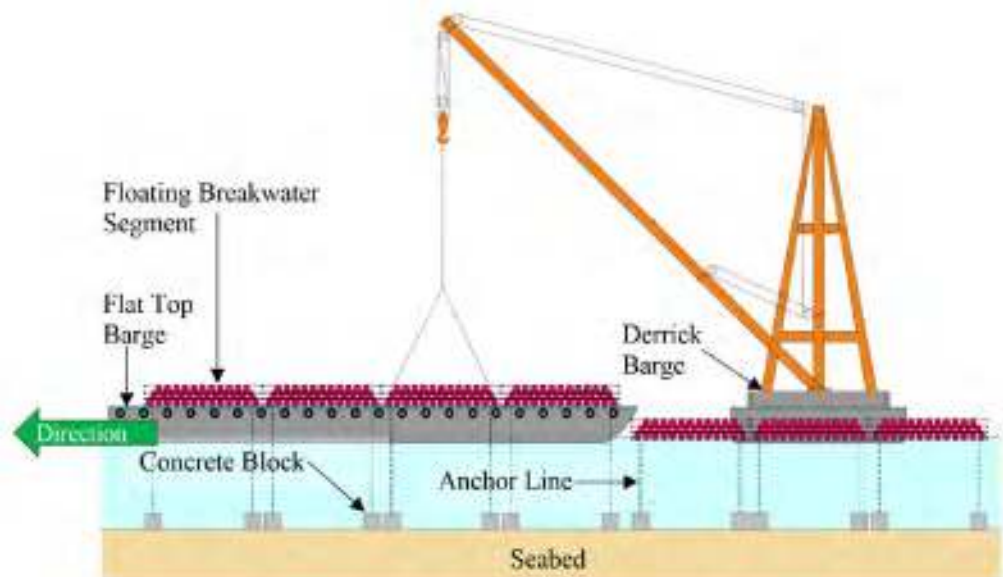


Figure 5: Installation methodology of the floating breakwater



Plate 4: a) Photograph of easy unloading of the floating breakwater units. b) Assembled floating breakwater segment. c) and d) Floating breakwater placement by Flat Top Barge

segments behind along with anchor blocks and is assisted by the Derrick Lighter crane. All temporary marker buoys are removed upon completion of floating breakwater installation. Divers and spotters are employed to carry out regular visual inspections and maintenance of the floating breakwater system. The breakwater segments remain afloat allowing for moderate deposition of debris and algae growth. Figure 5 indicates a schematic of the floating breakwater installation methodology and Plate 4 indicates photographs of the floating breakwater installation.



Plate 5: Aerial photograph of the P2 Project with the floating breakwater along the southern perimeter of the site

4.3 Observations

The vessel skippers and marine workers express genuine gratification and peace of mind upon installing the floating breakwater. The positivity observed is apparent at all working levels and provides the crew with confidence to carry out daily works on or above calmer waters. The direct benefit witnessed by workers are materials on vessels being less susceptible to sliding, tipping or spinning, and reduced risk of workers falling into water from vessel movements. Marine crane operators have easier response to material movement during lifting and workers are less prone to injury from moving mooring lines. The operational benefits observed include the reduction of mandrel breakage during band drain installation by marine methods and the reduction of line jerking on temporary bollards from moored construction vessels.

Albeit, the Contractor's and Designer's confidence is further reinforced after the site was subject to Super Typhoon Mangkhut on September 2018. Post typhoon geophysical surveys within the site boundary indicated negligible scouring of the temporary reclamation leading edge and the existing seawall with permanent rock armour removed. Minimal adverse stability effects were apparent on the environmental temporary environmental cofferdam. The floating breakwater not only remained intact and on the as-built alignment after the Super Typhoon, but minimized the overall damage to the site. The floating breakwater has thus enhanced the overall resilience of the Project during the typhoon season in 2018.

At the time of paper writing, a workable methodology is being prepared to monitor the performance of the installed floating breakwater during a severe typhoon. The use of wireless instrumentation, CCTV cameras on seawater level rulers and floating marker buoys are all considered to establish a safe and remote method of monitoring of the wave height to quantify breakwater attenuation ability.



Plate 6: Photograph of installed floating breakwater

5 CONCLUSIONS

Through genuine partnering and with the innovative spirit of the Contractor CRBC-BK JV, their Designer MIEL, the Employer CEDD and their Consultant AECOM, the first 250m long temporary floating breakwater in Hong Kong is successfully implemented for the construction of Contract “Tseung Kwan O – Lam Tin Tunnel - Road P2 and Associated Works”. In addition to safe working practices, administrative, engineering and protective measures, the confidence in current breakwater technology proactively spearheaded the provision of an additional floating breakwater to protect the site from wave effects. The extra merits of the floating breakwater relieve the need of fixed foundations, has a small footprint, is relatively low cost, is quick and easy to install, requires low maintenance and has good alignment flexibility. Also construction vessels are unscathed upon contact with the breakwater HDPE material.

The floating breakwater provides a comfortable marine working environment for workers by providing effective wave attenuation throughout the construction period thus enhancing certainty in safety and programme achievement. Furthermore, the floating breakwater remained intact during Super Typhoon Mangkhut on September 2018, minimized weather related damage on site and has greatly improved the resilience of the Project.

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Stability Improvement to a Loose Fill Slope in a Housing Development Project

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ABSTRACT

The stability upgrading of loose fill slopes has long been a challenge of slope engineering practice in Hong Kong and worldwide. The application of soil nails to slope upgrading works in Hong Kong can be dated back to the mid-1980s (Watkins & Powell 1992). In its early stage of application in Hong Kong, the soil nailing technique was limited to existing cut slopes and temporary cuts in hillside terrains. As to the stability improvement to loose fill slopes, not until recently the local practice has been in need of recompaction of top 3-meter of the loose material on the slope (Ng & Lumb 1980) with or without additive of cement to enhance the shear strength of compacted soils (Mitchell 1976). The finished slope surface would be protected further by spray concrete against surface erosion or by hydro-seeding and planting of trees and shrubs for the purpose. In recent years, there have been great public concerns about the slope recompaction works which have to remove existing trees on the slope, very often, most of them mature trees. Geoguide 7 to soil nailing technique for stability improvement of loose fill slopes was thus brought about by GEO in 2008 (GEO 2008). This paper presents a case of the stability improvement to a loose fill slope in a housing development project site, where both techniques of soil nailing and recompaction of soil cement fill were adopted. Finally, discussion and conclusion on the design and construction practice of the improvement works to the fill slope will be given.

1 INTRODUCTION

The subject of this paper addresses the geotechnical challenges and measures adopted in the design of slope upgrading works in a housing development project. The project site comprised a number of cut and fill slopes between formed platforms. The subject slope was a large soil/rock slope, stretching some 220m across the site from South to North, with a soil slope portion varying in height from 48m at its southern end whereas its rock slope portion about 22m in height at the other end to the north. The soil slope portion measured an average slope gradient of 32°, and there were a good number of mature trees on the slope while the rock slope portion stood bare and almost sub-vertical. There was a transition zone in the middle third between the two portions (Figure 1).

2 GROUND AND GROUND WATER CONDITIONS

At about 70m from the northern end, the slope formation transited from mainly a rock cut gradually into a soil slope at the southern end. The soil slope portion had a thin layer of fill to a maximum thickness of about 3m overlying the in-situ material of decomposed granite to various degrees of weathering. The bedrock was described in borehole logs as strong, light grey, spotted black, slightly decomposed, medium grained Granite with joints closely to medium spaced, smooth planar and slicken-sided undulating, extremely narrow, iron stained, kaolin coated, dipping 40 to 60 degrees. The fill comprised

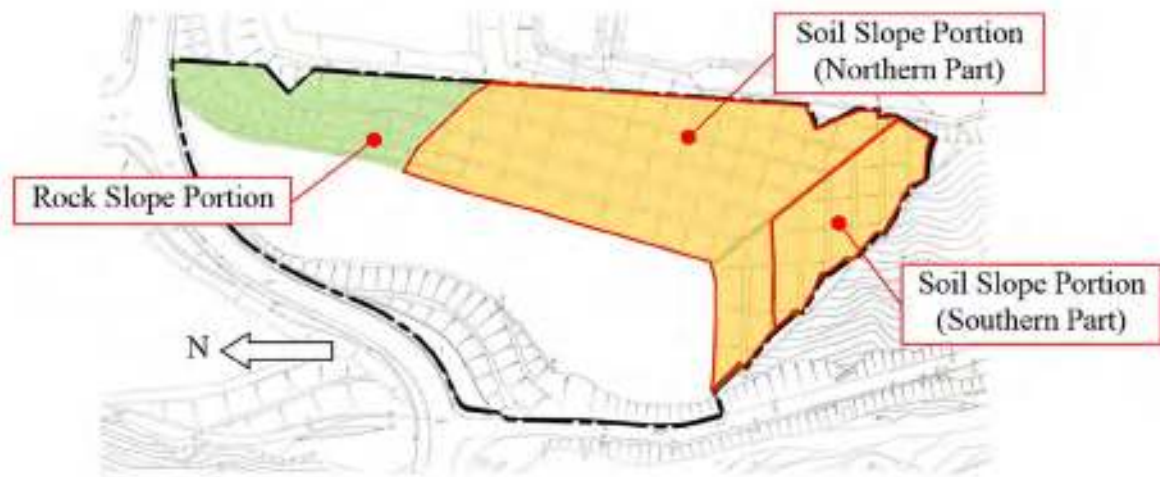


Figure 1: The layout of the subject slope

largely coarse gravels with occasional cobble sized moderately strong rock fragments that made the top soil barren and unsuitable for slope greening.

At the time of ground investigation, groundwater was encountered at about 2.4m below ground at a drillhole near the crest of the soil slope portion near the southern end whilst the highest and lowest measured ground water levels at the same drillhole were registered at 0.75m and 9.9m below ground respectively during groundwater monitoring over two wet seasons.

3 PROPOSED SLOPE UPGRADING WORKS

Given the existing site constraints and a design requirement of factor of safety (FOS), viz a $FOS > 1.4$ for stability of slopes (GEO 1984) that may affect or be affected by the development, soil replacement and slope stabilization measures were found warranted. The design of slope upgrading works for soil slope portion adopted the following schemes:

- (1) Scheme 1 (Northern Part): Compacted soil cement fill being applied to replace existing fill in slope portion where no trees on the slope are required to retain, and
- (2) Scheme 2 (Southern Part): Where trees were in clusters or stood reasonably close to one another, soil nails being applied between trees to underlying competent ground.

Figure 1 shows location of northern and southern parts. Figures 2 and 3 illustrate the proposed schemes of upgrading works aforementioned.



Figure 2: Scheme 1 (Northern Part) – Replacement of existing fill by soil cement fill



Figure 3: Scheme 2 (Southern Part) – Installation of soil nails with concrete grillage beam

For scheme 1 where a maximum depth of top 3m fill, or to top of in-situ soil whichever would come first, the top soil was to be replaced by compacted soil cement fill. Tree pits of 2m x 2m x 1m (averaged) deep at 8m spacing were provided for trees on the finished slope (Figure 2). Figures 4 and 5 show the typical arrangements of the soil replacement scheme. For the scheme 2 where existing trees stood in clusters or closely spaced, soil nailing should be carried out in between tree clusters so that the grillage beams could run between nail heads in an irregular manner leading to a free-style grillage beam system (Figure 3).



Figure 4: (left) Construction of tree pit in soil cement fill slope &
Figure 5: (right) Typical section of tree pit to facilitate slope greening

4 GEOTECHNICAL PARAMETERS

The geotechnical parameters adopted for the slope works are given in Table 1 below. The parameters were derived from site specific ground investigations and they were found lying within typical range of values for materials commonly used in the local practice (GEO 1993).

Table 1: Geotechnical parameters of soils and rocks

Materials	Bulk weight, γ (kN/m ³)	Cohesion, c' (kPa)	Friction angle, ϕ' (degrees)
Existing Fill	19	0	30
Compacted Fill	19	0	38
Completely Decomposed Granite	19	6	37
Highly Decomposed Granite	19	7	38

As the existing fill contained a lot of coarse grained materials which were considered not suitable for slope greening, replacement of the existing fill by compacted soil cement fill was carried out to a maximum depth of 3m. For this slope, we intended to design all finished slopes so that they would environment friendly to the surroundings. Thus, the soil cement fill was so designed that its engineering properties should be similar to that of CDG, viz $c' = 6\text{ kPa}$ and $\phi' = 37^\circ$ as shown in Table 2 below.

Table 2: Composition of soil-cement mix (by mass)

	Portland Cement	Sand	Inorganic soil
Proportions	1	3	12
Specification	BS-EN197-1	BS1200	Free of organic matter and not more than 30% of soil particles passing a 63 μm BS test sieve

Undisturbed samples were taken from the finished slope for triaxial test in order to verify the engineering properties of the compacted soil cement fill.

5 RESULTS OF VERIFICATION TESTS ON STRENGTH PARAMETERS OF COMPACTED SOIL-CEMENT FILL AND ON SOIL NAILS

Consolidated undrained triaxial compression (CUS) tests were carried out to verify the design parameters of soil cement fill. The results indicated a good agreement between the designed parameters and those derived from the test results as shown in the s' - t plot in Figure 6 below.

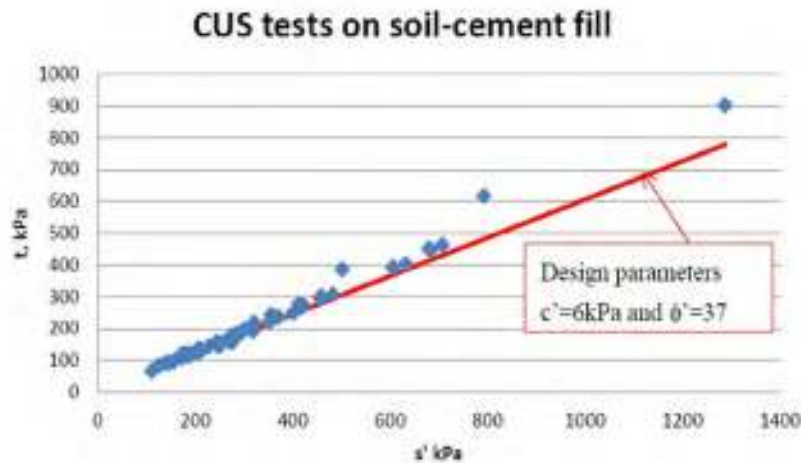


Figure 6: CUS test results against design shear strength parameters adopted for soil cement fill

As to the soil nailing works, pullout tests were carried out on trial soil-nails at selected locations to verify the design assumptions and the workmanship of contractor before commencement of soil nailing works. Trial soil nails of a total number equal to 2% of the total number of working nails were tested according to Geoguide 7 (GEO 2008).

6 DISCUSSION

The slope upgrading works in this project faced a number of challenges. Firstly, the site formation works had left behind a thin layer of loose fill spread over the slope. Secondly, at the southern end on the upper portion of slope surface there were grown-up trees that needed to be maintained during and after works. Thirdly, large quantities of gravel, cobble and rock fragments in the existing fill made the fill recompaction impractical and difficult to comply with the specification, i.e. 95% degree of compaction. Lastly, the fill appeared to be too barren and unsuitable for slope greening. So, soil cement fill which mixed of imported Completely Decomposed Granite (CDG) and Portland cement was adopted to replace the existing fill because CDG is good in engineering properties and plenty in local supply, it is also considered environmental friendly for growth of many native species of plants on the slope and thus conducive to slope greening in the long term.

However, given the existing slope gradient and required safety margin of the finished slope profile, the CDG fill needed to mix with cement as additive to improve its shear strength. Use of cement and other chemical as additives for soil improvement has been studied for decades by many researchers for various objectives (Mitchell 1981). Attainment of good quality of the results required deliberate control over the design mix and supervision of workmanship on site. The design mix of soil cement fill used for this project had undergone a process of site-trials. The successful verification tests carried out with soil samples taken from the finished slope confirmed the design mix and workmanship to accomplish the slope upgrading works. It can be illustrated by the photo of the upgraded slope in Figure 7 below taken about 2 years after the works.

Besides, it has been proven a successful attempt in the use of an irregular grillage beam system which linked the heads of soil nails in such a way to avoiding the existing trees on the slope. This successful attempt added aesthetic value to the finished slope, viz the structural features blending in harmony with the environment. It can be illustrated by the photo in Figure 8 below which was taken at a time about 2 years after the works.



Figure 7: Growth of vegetation on the slope about 2 years after the slope upgrading works



Figure 8: Close look at the free-style grillage beam system on the soil-nailed slope portion with existing trees

Existing trees can be preserved as far as possible and afterwards there also came in native shrubs to grow on the slope surface between grillage beams. A 150mm up-stand was provided, protruding from the edge of grillage beam for retention of top soil cover as a nursery bed for planting of more shrubs and growth of native species from nearby to come in later. (Figure 9)



Figure 9: A new design initiative to the grillage beam which included a 150mm up-stand for retention of top soil as a nursery bed

7 CONCLUSION

This paper presented a case of stability improvement works to a loose fill slope that made use of soil cement fill. The strength of the fill was enhanced for slope stability with addition of a small amount of cement. In addition, tree pits were provided inside soil cement fill due to the need of slope greening. A special arrangement of grillage beam system for soil nail heads on the slope was designed to avoid any felling of existing trees, which included a new design initiative (150mm up-stand) to the grillage beam design to facilitate slope greening.

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Application of Real-Time Remote Monitoring on Flexible Barrier

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ABSTRACT

Urban development and expansion in Hong Kong are gradually encroaching to the edge of steep natural hillsides. In the face of climate change leading to more frequent extreme weather events, the susceptibility of these natural hillsides during intense rainfall to landslides is increasing. Flexible barriers as landslide debris-resisting fences have become the preferred option of mitigation measures for natural terrain hazards in Hong Kong, as they are more adaptive and buildable on steep hillsides. Subject to boulder or debris impact and deposition, flexible barriers are designed to withstand these events. Geotechnical monitoring has been used to monitor the performance of the barriers after installation, and these data act as an important element of understanding the service life of the structure and its maintenance requirements. Traditionally, geotechnical monitoring has been performed manually, which can present restrictions of access, adverse weather, time-delay, consistency, frequency, and reliability.

Automating the monitoring systems with wireless networked sensors not only overcomes these issues, but also offers additional advantages, such as the ability to monitor several sites in one location and provide real time data feedback. The wireless Tilt and Vibration (T&V) sensor and 3G gateway is one of the solutions. They are typically installed on or around sensitive building structures during excavation works, but in our case study, they were installed on flexible barriers for monitoring tilt on posts and detecting impacts when debris strikes the barrier, which indicates deterioration of the barrier by accumulated debris, and requirement of maintenance. The sensors are able to be placed in remote locations, and transmit sensor alerts direct to mobiles immediately when the impacts or tilts meet the pre-set threshold, as well as regular server uploads. This paper discusses the application of real-time remote monitoring on flexible barrier with regards to the performance during wet season, the ability of potential early detection system and an exploration of their use for long term maintenance monitoring.

1 INTRODUCTION

Climate change and extreme weather events are regarded by many to pose a significant risk to the terrain of Hong Kong. With its long history of natural terrain mitigation, Hong Kong is regarded as a leading force in innovative solutions to arrest and mitigate against landslides and as a highly developed and increasingly digital urban metropolis, it is a natural progression to utilize cutting edge technology to aid in tackling these threats. Whilst the use of automated and real-time monitoring technology is not a fresh idea in the construction industry, the use of such monitoring systems as Automated Deformation Monitoring System (ADMS) and tilt and vibration sensor system (T&V), is still regarded to be in its infancy in terms of natural terrain monitoring. As such devices become increasingly cost effective,

user friendly and reliable through their wide adoption on construction sites, their effectiveness and ability to enhance monitoring performance on natural terrain mitigation measures requires further investigation. In this paper, the application of automated and real-time monitoring technology with regards to flexible barrier maintenance will be discussed in relation to a case study and the technical challenges encountered.

2 APPLICATIONS IN FLEXIBLE BARRIER

2.1 Application of Flexible Barrier and Current State of Maintenance Procedure

Flexible barrier systems are widely adopted as passive mitigation strategy for natural terrain hazards such as rockfall, landslides and debris flows, as the system is able to capture large deformation and makes the barrier suitable to mitigate against the impact loads of landslide materials (Volkwein et al., 2011). The systems consist of metallic nets with chain-link meshes, high-tensile steel ropes, brake rings and structural steel posts. Plate 1 demonstrates the structure of a typical flexible barrier system. Since 2000, the Hong Kong Government has paid a great attention to natural terrain hazards, and hundreds of steel flexible barriers have been erected on natural hillsides as a protective measure to mitigate landslide risk. However, after the successful implementation of these steel flexible barrier systems, they have seldom been maintained and inspected during the past decade due to the inherent remoteness of their locations. The condition surveys of the steel flexible barrier have typically been conducted manually by sending out engineers for inspection if a landslide was reported.



Plate 1: Typical Flexible Barrier in Hong Kong

2.2 The Wireless Tilt and Vibration Monitoring System and Application

Automating the monitoring systems with wireless networked sensors is proposed as a means to overcome site inspection and maintenance issues in remote and inaccessible areas. Following current trends in the industry, wireless Tiltmeters and Vibration (T&V) sensors have become widely adopted for structural health monitoring, expansive area and long distance monitoring, natural slopes monitoring, as well as barrier impact detection. The T&V sensors are high precision wireless MEMS sensors with built-in temperature compensation. Working in a similar way to sensitive inclinometers that measure

the changes of positions from single or dual axes, they are typically attached on the sensitive structures or utilities surface, to provide indication of structural stability or deterioration risk of the structure (Chan et al., 2017).

The T&V sensors are able to provide real-time precision tilt readings (in degrees), impact detection (in g) and temperature readings (in Celsius). In order to place the sensors in remote areas, leveraging with a 3G gateway is required, meaning alerts are transmitted within seconds and regular readings are uploaded on a specific scheduled basis. Moreover, the sensors have configurable tilt and impact thresholds that allow automatically generate alerts when a set value is exceeded. The 3G gateway will then deliver sensor alerts immediately to the connected mobile numbers through SMS and transmit the data to a designated FTP server. The sensor alert data can be configured to contain both pre- and post-alert data for future trend analysis of the steel flexible barrier.

In term of applications in flexible barrier, the wireless T&V sensors are installed on the steel posts in order to monitor the tilting condition of the posts and detect impacts when debris strike and accumulates at the steel flexible barriers (Plate 2). One of the features of these wireless T&V sensors is to connect with a mesh network ranging in size from one to a hundred sensors and operating in bi-directional mode, in which each post can be installed with the sensor and communicate with other sensors on multiple flexible barriers.



Plate 2: Wireless Tilt & Vibration Sensor Installed on the Steel Post

2.3 Advantage of real-time monitoring over traditional method

The 'Internet of Things', IoT, is a buzz word in most circles and the construction industry is no different. As data production, transfer and delivery becomes more digitized, time is saved if each stage is automatically received in a digital state for storage and manipulation. Traditional data collection would require additional stages to convert and upload the data causing delays and additional resources drain. By automating data collection in a digital format, every stage from design, construction and end use, can access the data in one shared format. Although automated and real-time monitoring technologies are widely applied in monitoring the performance of structures in construction stage, it has not been popularize in term of maintenance stage, especially in the application of flexible barrier.

In comparison to the traditional method of conducting surveys of the steel flexible barrier manually

after landslide was reported, these T&V sensors were able to reduce manpower for inspecting steel flexible barriers that were not deteriorated. In addition, due to the automated process of the T&V sensors, the possible drawbacks of human errors in manual reading during inspection are negated by enhancing the consistency of regular data reading.

Traditionally, the condition surveys of deteriorated steel flexible barrier were done retroactively once a landslide was reported. The wireless T&V monitoring systems shown to be able to withstand under adverse weather, including rainstorm and typhoon, therefore, once landslide occurred during adverse weather, these wireless systems allows impact load to the flexible barrier to be captured in real-time and generate alerts when set limits are exceeded; this ultimately allows a more active approach for inspection to the barrier and maintenance if necessary and also further data for assessment of the behavior of the barriers. Thus, real-time monitoring shown to be a cost-effective approach comparing to the traditional method.

3 IMPLEMENTATION OF T&V MONITORING SYSTEM SITE TRIAL

3.1 Site Background

The T&V monitoring site trial was proposed as an additional technical study associated with the steel flexible barriers condition survey and enhancement works. The aim of the site trial is to investigate the connectivity of the T&V sensor at remote areas and the durability of the sensor under adverse weather conditions in order to determine the feasibility of carrying out long term monitoring on steel flexible barriers in natural terrain.

The monitoring sensors were installed on the selected steel flexible barriers to be maintained under the works contract. The selected site is located at the remote foothill of a valley in Lantau Island where several village houses are located at the toe of the natural hillside. Three steel flexible barriers from top to bottom were erected at the Site with 28m, 37m and 22m in length. The distance on plan between lower barrier and upper barrier is approximately 50m. Moderately dense tree canopy is covering the three flexible barriers.

Four T&V sensors were installed near the top of the steel posts at the upper barrier and the lower barrier taking into account the potential debris trajectory and the sensitivities of the deformation received from steel post during an impact. A 3G gateway was installed on the middle barrier for better connection to receive the data collected from the sensors and transmit to the FTP server. Figure 1 presents the location of the T&V sensors and the 3G gateway.

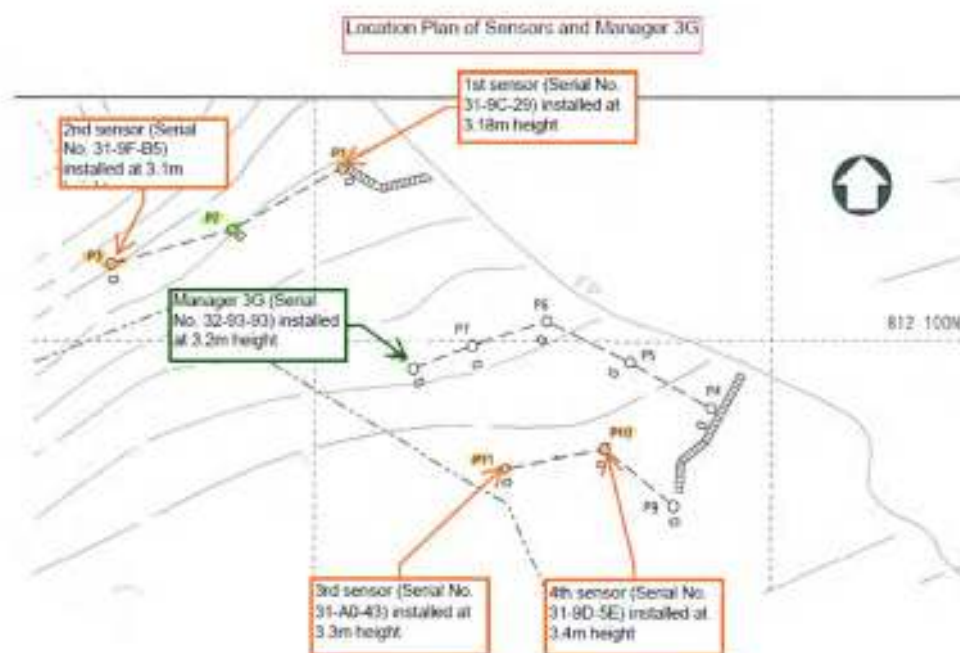


Figure 1: Location of the T&V Monitoring Sensor System

The site trial was carried out between July and August 2019, where heavy rain storms and landslides were envisaged to be most frequent. Within the monitoring periods, the T&V sensors recorded the pitch and roll at regular time intervals to monitor any deformation. In addition, the sensors would also record the reading once a sudden impact exceeding 250mg was detected.

Due to the consideration of the relatively large deflection angle to be encountered at a higher position of the steel post during impact, the “Alert level” was set to an angle of 5° tilt from the original pitch and roll axes, for an instant alert message.

3.2 Weather Condition during the Monitoring Period

Weather events data from Hong Kong Observatory could be one of the references points related to the behavior of monitoring records. Rainfall intensity, wind speed, Tropical Cyclones Warning Signals, Rainstorm Warning Signals, Landslip Warning, Cold and Hot Weather Warning will be considered as the influence factors.

The weather in July was generally cloudy and showery with a total of 341.1mm rainfall was recorded during the month. The rainguage located at Ngong Ping Fresh Water Reservoir recorded the monthly rainfall was 274mm. Heavy showers and squally thunderstorms with Amber rainstorm warning signals were hoisted on the 2 and 15 July. The maximum daily rainfall near the Site was 71.5mm and was recorded by the rainguage on 14 July. Tropical cyclones warning signals no. 1 and 3 were also hoisted on 17 – 18 July and 24 July.

The weather in August 2018 was wetter and gloomier, as it was affected by the tropical cyclone Bebinca, and tropical cyclone warning signal no. 1 and 3 were hoisted in between. The recorded monthly rainfall was 615.1mm. Thirteen amber rainstorm warning signals and two red rainstorm warning signals were recorded during 10 – 29 August. Landslip warning was also hoisted followed by the heavy rain between 29 and 30 August. The rainguage in the vicinity of the Site recorded the monthly rainfall was 707 mm and the maximum recorded daily rainfall is 73.5 mm on 19 August. The rainfall data in July and August is shown in Figure 2.

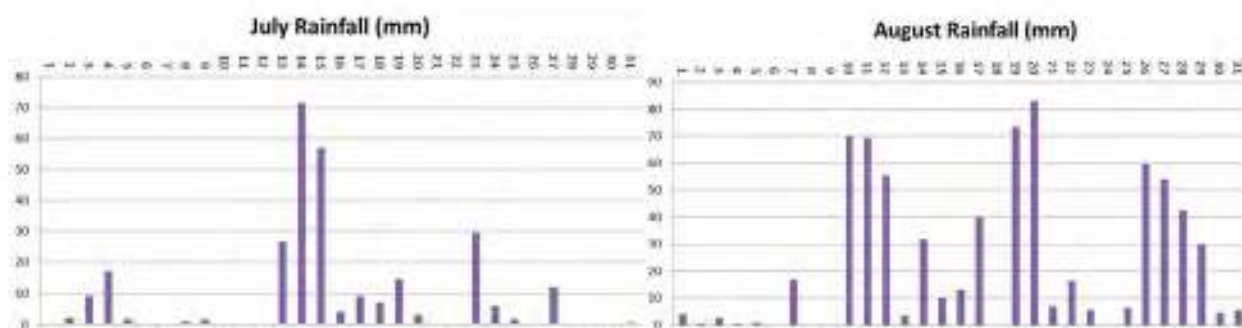


Figure 2: Rainfall data in July and August

3.3 Results and Findings

During the monitoring period, no alert messages were received, so we can presume that no significant landslide and impact or deformation was performed. However, several alerts were received on 17 July 2018 and 8 August 2018 due to the repositioning of the sensors by our contractor. Plate 3 illustrates the photograph of the repositioning of the sensor by the contractor. Figure 3 presents the monitoring data obtained in July and August 2018 respectively. The data indicates the sensors operated as usual during the adverse weather condition although no abnormal deflection or impact was picked up during the monitoring period. These monitoring results had been verified by manual site inspection after adverse weather.

In order to investigate the factors which could influence the result of the sensor, results from Figure 3 is studied in closer detail and compared to the records of temperature, wind speed and rainfall intensity. Direct comparison between two graphs by means of data trends was used in the following analysis. By comparing the wind speed and rainfall recorded with the tilting monitoring data, the trend of those

graphs seems to be uncorrelated. In addition, Figure 4 shows the pitch angle and the temperature of the sensors in July and August. The results indicate that an increase of tilting degree occurs with temperature increases, due, potentially, to the thermal expansion and contraction of the post.



Plate 3: Photograph Showing the Repositioning of the T&V Sensor

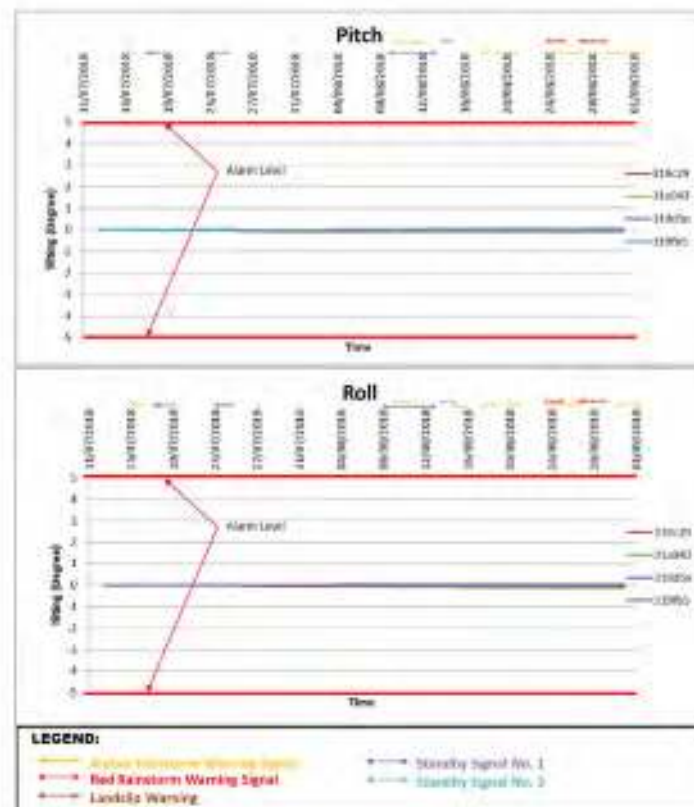


Figure 3: Monitoring Data in July and August

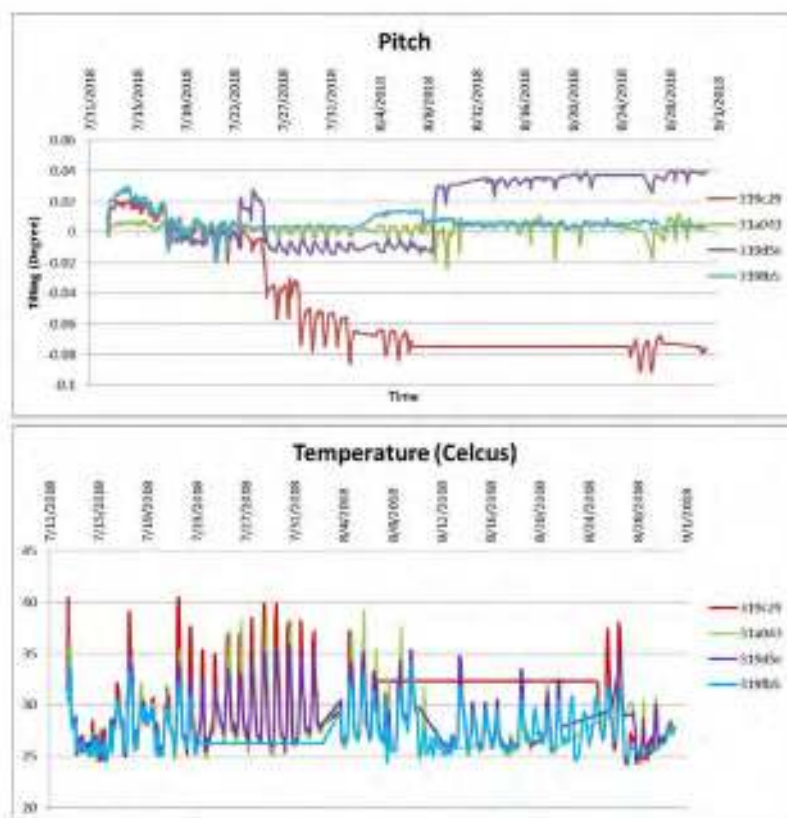


Figure 4: Comparison of Monitoring Record and Temperature Data of July 2018

4 RECOMMEDATION FOR FURTHER STUDY

As steel flexible barriers become widely adopted as mitigation measures to natural terrain hazards all around the expanding urban area, the maintenance of flexible barriers becomes increasingly complicated and resources consuming. Therefore, engineers endeavor to seek automation and remote sensing techniques to overcome the arduous and inefficient manual operation.

The site trial of T&V monitoring on flexible barriers proved automation and remote sensing solution could enhance resource management. The real-time monitoring enables the engineer to understand the condition of the flexible barrier without carrying out inspection in densely vegetated terrain. The wireless network is provided using a SIM card giving flexibility to placement of the sensor at remote areas where signals may vary for different telecommunication service providers. The compact size and 'plug-and-go' nature of the T&V sensor mean the sensors can easily be installed on higher portions of the flexible barrier to gain better network coverage, as well as minimize the chance of damaging the sensor upon impact.

The current site trial only proved the feasibility of adopting T&V sensors on monitoring flexible barriers and the durability of the monitoring devices. However, since no debris strikes were recorded on the flexible barriers during the site trial period, the monitoring data obtained is insufficient to deduce the adequacy of the assumed alert threshold. In view of that, continuing the real-time monitoring strategy to obtain more data is highly recommended. Analysis of the data suggests the T&V sensors are very sensitive that they often pick up minor degrees of deflection which is not due to destructive impacts. Therefore, apart from carrying on monitoring, further study on the application of the sensor, specifically on the degree of sensitivity to be adopted on the sensor and the establishment of an appropriate "Alert" level that triggers further manual actions, should also be considered. It is also suggested to investigate any deflection patterns and trends to identify the different types of impact that may be encounter besides landslide. To achieve this aim, carrying out field tests such as hammering the flexible barrier to simulate different impact situation is recommended. The hammering test is aim to narrow down the range of energies to consider as a potential destructive impact, hence to establish a better alert threshold. Besides hammering, material including various sizes of boulders, steel rods as to simulate

tree trunk will also be considered as test materials for the impact test in order to increase the accuracy of the simulation.

5 CONCLUSIONS

The use of automated and real-time monitoring devices to enhance flexible barrier maintenance has been presented. With reference to the increasing demand to unite technological advances with real world solutions as well as reducing labor intensive actions, the effectiveness of these devices to provide accurate and meaningful data are discussed. This most recent case study demonstrates that the equipment is capable of handling the technical challenges posed by the natural terrain and is equally able to stand up to the elements during weather events which represent a crucial window for flexible barrier operation. Despite limited significant data collected during this current trial, continuous data collection provides the opportunity of both real time assessment of critical events and also a long term assessment of trends in the condition of the flexible barrier. Further studies into identifying signatures and trends for typical readings and a suitable range of sensitivity will be explored.

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Adaptive updating of soil properties through monitoring data for improved prediction of excavation response

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ABSTRACT

This paper presents an adaptive model updating approach for deep excavations, which considers various sources of uncertainty that lead to discrepancies between predicted and actual excavation responses. The approach utilizes field monitoring data to update the soil stiffness and strength parameters for multiple soil layers. Based on the updated parameters, the subsequent excavation responses and levels of uncertainty are represented by prediction intervals, continuously refined as the construction progresses. Furthermore, to reduce the high computational demand commonly encountered in updating algorithm, the approach incorporates machine learning techniques including surrogate modelling and principal component analysis. The proposed adaptive updating approach can be readily incorporated into commercially available finite element or finite difference packages, and is illustrated through re-analyses of the multi-stage braced excavation at the Tsuen Wan West station. The case study shows that the estimated prediction intervals can encapsulate the actual response, meanwhile providing quantitative indicators regarding the levels of uncertainty for future construction stages. The measurement data is also shown to effectively reduce uncertainty in the soil parameters, manifested as the narrower prediction intervals as the construction progresses. The approach provides an efficient modelling tool to facilitate data-driven decision making based on field measurement data, thereby contributing to the quantitative framework for the observational method.

1 INTRODUCTION

The designs of excavation and lateral support systems are routinely performed by deterministic analyses, where a set of ‘moderately conservative’ design parameters are adopted to predict the response of retaining walls and the nearby ground and structures. In the selection of design parameters, variations in the soil properties, even within the same stratum, are only incorporated implicitly, and the uncertainty in geotechnical parameters is one of the leading causes for discrepancies between predicted behavior and actual system response (Baecher & Christian 2003). Recent developments in probabilistic approaches (e.g. Lo & Leung 2017; Leung & Lo 2018) allow geotechnical uncertainty to be explicitly and quantitatively incorporated into the analysis process, while Bayesian methods present the mathematical tools to incorporate new information (such as monitoring data) to refine our understanding of the uncertain and variable site conditions. These tools also resonate with the concept of ‘observational method’, outlined by Peck (1969), which emphasizes the needs to incorporate new knowledge of site conditions as construction progresses and revise the original assumptions if necessary. The method can be pertinent to deep excavation projects, as fine adjustments of support layout may be possible during construction, based on monitoring data obtained from earlier stages.

Bayesian model updating provides a quantitative framework for the observation method. Uncertainty in soil parameters is initially specified as a probability distribution known as prior distribution, which can be updated through the monitoring data, to obtain the posterior probability distribution. According to Bayes’ Theorem:

$$f(z|y) = kf(y|z)f(z) \quad (1)$$

where z is the vector of soil parameters, and y is the vector of monitoring data. $f(z)$ and $f(z|y)$ are the prior probability density and the posterior probability density of soil parameters, respectively. $f(y|z)$ is defined as the likelihood function that depends on the discrepancies between measurements and predictions. These discrepancies are mathematically represented by $y-g(z)$, where g is the predicted response (based on parameters z), usually obtained by finite element or finite difference methods. In Eq. (1), k is a normalizing constant for the posterior distribution.

From the engineering perspective, the quantities of interest are the mean and covariance of the updated soil properties $z|y$, which represent the best estimates of updated parameters and the associated uncertainties. Usually, the covariance will reduce after the updating process, when the monitoring data effectively reduces the uncertainty in soil parameters. In addition, making use of the updated parameters, the predicted excavation response for subsequent construction stages can be refined. A 'prediction interval' can also be defined to encapsulate the 'likely' response with various levels of confidence. Normally, these prediction intervals gradually become narrower after several updating stages, due to the reduced uncertainty in the soil parameters.

There are several challenges when applying the Bayesian methods in geotechnical engineering. Previous applications to deep excavations typically made use of the measured maximum wall deflection or maximum settlement to update geotechnical parameters (Juang et al. 2013; Wu et al. 2014), without fully utilizing all the data from continuous measurements of ground settlements or wall deflection (e.g., inclinometer data along the entire wall depth). This may be attributed to the large number of measurement points involved, and hence a long vector y , that renders the formulation of likelihood function very complex. Furthermore, evaluation of the posterior probability involves repeated simulations of finite element or finite difference model for the likelihood function, which can be computationally demanding. This paper will address the above issues by machine learning techniques – principal component analysis will be adopted to extract the important features of wall deflection and soil settlement, significantly reducing the size of y vector; meanwhile, surrogate modelling can approximate the finite element or finite difference models by a function of a simpler form, which is much quicker to evaluate. These machine learning techniques are incorporated into the workflow of Bayesian model updating for multi-staged excavation projects, to be presented in Section 2. Section 3 illustrates its implementation to an excavation project previously reported by Pickles et al. (2006).

2 WORKFLOW OF BAYESIAN MODEL UPDATING

Figure 1 illustrates the workflow of the proposed Bayesian updating approach for deep excavation projects. The workflow is divided into the design stage (prior to construction) and construction stage. While the key steps are briefly described here, the mathematical details can be found in the corresponding references and are illustrated in the case study presented in the next section. In the design stage, the first step is to specify the possible variations of parameters for the soil layers. Such variations may be represented by the mean and coefficient of variation (CV). These can be estimated from ground investigation data, or from values suggested in the literature (e.g., Phoon and Kulhawy 1999). Next, multiple combinations of soil parameters can be generated, with the project geometry and load conditions simulated by the finite element or finite difference package, to obtain the corresponding wall deflections and soil settlements. Important features of the wall deflection and soil settlement are then extracted using principal component analysis (PCA). Detailed implementations of PCA can be found in references such as Jolliffe (2002). Subsequently, surrogate modelling can be adopted to establish the relationship between soil parameters and the extracted features. In Eq. (1), the model predictions g can then be obtained by the surrogate model. The surrogate model adopted in this study is known as polynomial chaos expansion (Ghanem & Spanos 1991). Other types of surrogate model, such as artificial neural network or support vector machine, are equally applicable under this framework. Based on the surrogate model, sensitivity analysis (Sobol' 2001; Lo & Leung 2018) can be performed to identify the most influential parameters to the excavation response, which should be updated for the subsequent stages.

During the construction stage, suppose the wall deflections and ground settlements at Stage i

are measured. Features of the measurements are first extracted using PCA to form the vector y . The posterior density of soil parameters, i.e. $f(z|y)$, is then evaluated using a technique called Markov Chain Monte Carlo (MCMC) method, which allows the mean and covariance of updated parameters to be estimated. An implementation guide in MCMC for geotechnical engineering can be found in Juang and Zhang (2017). Based on the updated parameters, prediction interval for excavation response at Stage $i+1$ will be refined. Finally, for multi-staged updating, the posterior density at Stage i will be treated as the prior density at Stage $i+1$. This allows the estimates of soil parameters and the uncertainty to be continuously refined through repeated updating. As a result, the prediction intervals become narrower (i.e., less uncertainty) as construction progresses.

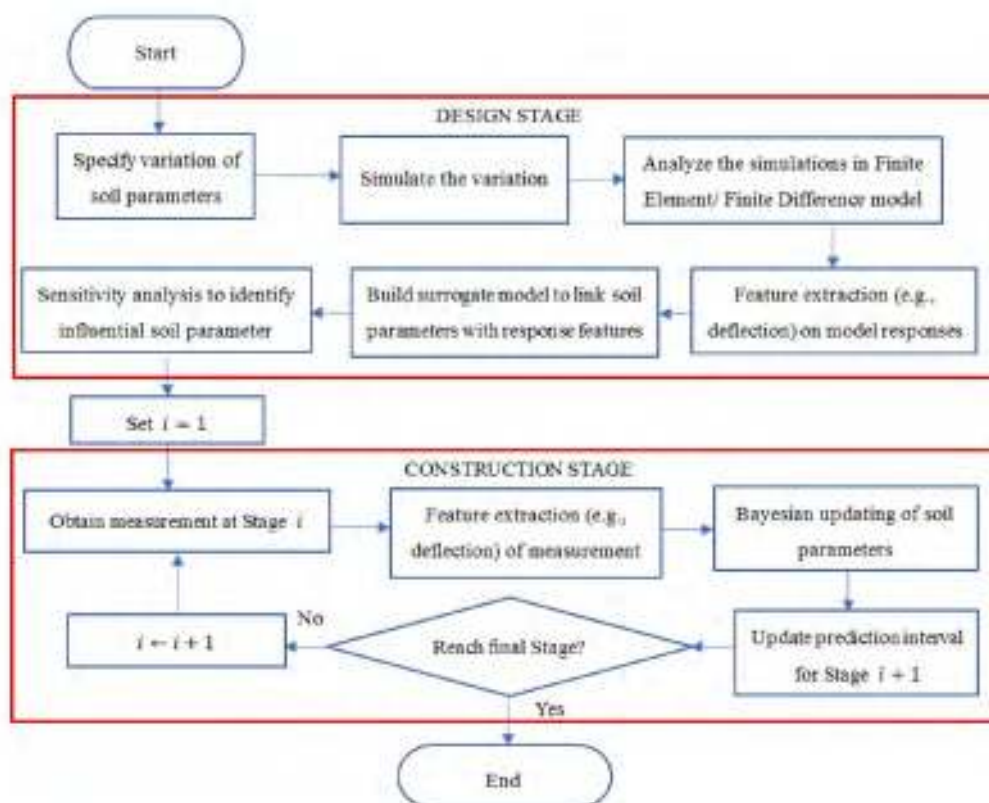


Figure 1: Workflow of Bayesian model updating for multi-staged deep excavation projects

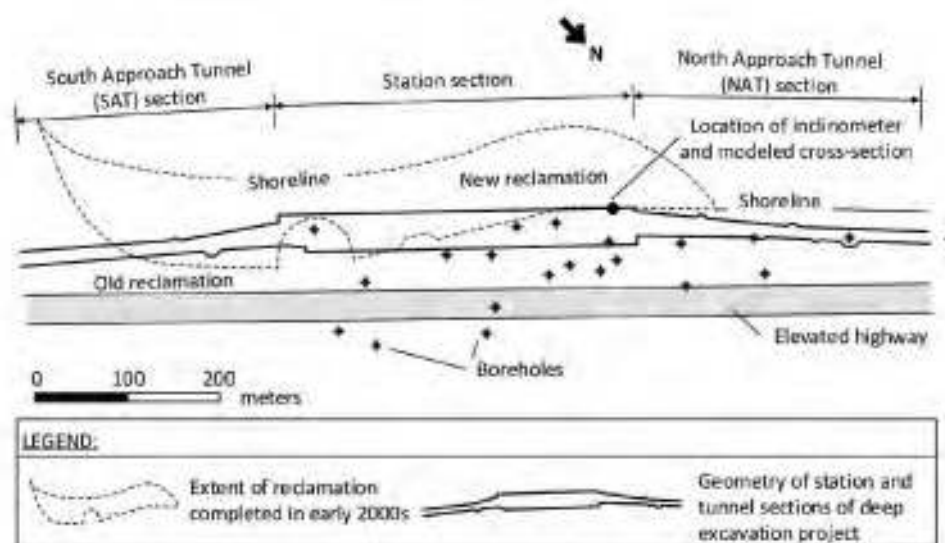


Figure 2: Layout of the excavation at Tsuen Wan West and location of the selected cross-section with inclinometer and settlement measurements (adapted from Pickles et al. 2006)

3 EXCAVATION CASE STUDY OF BAYESIAN MODEL UPDATING

3.1 Ground conditions

Figure 2 shows the layout of the project which is re-analyzed in this study, while details of the project background, site conditions, monitoring data and deterministic finite element analyses have been reported in Pickles et al. (2006). The project site is located in Tsuen Wan West area in Hong Kong. It consists of a 400-m long underground station and 600 m of cut-and-cover tunnels, and was constructed in the early 2000s. In this study, a section of the deep excavation on the seaward side of the station is investigated. The ground strata at the seaward side consist of 18.5 m of new reclamation fill, completed shortly before the construction of station. The fill overlies a 2-m layer of alluvium (ALL), followed by completely decomposed granite (CDG) which is approximately 3 m thick. The rockhead level is at -19 mPD, or about 24.5 m below the ground surface. The water table is at +2 mPD.

3.2 Excavation design and monitoring

The lateral support system at the station consisted of reinforced concrete diaphragm wall of 1.2 m thick, extending from +4.5 to -25 mPD, while the bottom 6 m of the wall was embedded in rock. The excavation depth was from +4.5 to -12.5mPD, with two levels of temporary struts at +1.5 and -7.5 mPD. The excavation sequence is outlined in Table 1. The steel struts were double UB 610×324×174 sections with lateral spacing of 7 m. During the excavation, the groundwater level is lowered to 1 m below the excavated level inside the cofferdam, and the groundwater level is maintained at +2mPD outside the excavation. The wall deflection is monitored through inclinometer readings, which are measured at 0.5-m intervals, from +4 to -24.5 mPD. Meanwhile, the nearby ground settlement is monitored through five monitoring points installed behind the retaining wall. Therefore, the total number of measurement points is 63. The deflection and settlement profiles measured during excavation are extracted from Pickles et al. (2006), and will be presented in later sections with the analysis results.

Table 1: Construction sequence of the Tsuen Wan excavation case

Stage	Reduced level of strut (mPD)	Excavation level (mPD)
1	Nil	+0.5
2	+1.5	-5
3	Nil	-8.5
4	-7.5	-12.5

3.3 Probabilistic analysis prior to Bayesian updating

Pickles et al. (2006) also presented the deterministic back analyses of the same section using the finite element software PLAXIS, with the Mohr-Coulomb model and then Hardening Soil model adopted to represent the constitutive relationships. In this study, the numerical model of the same section is simulated using FLAC, which is a finite difference software. The geometry of the model is similar to that by Pickles et al. (2006), while this study focuses on the probabilistic analyses and Bayesian updating aspects. Figure 3 shows the FLAC mesh grids with the soil layers at construction Stage 4. A shear hardening soil constitutive model ('Chsoil' model) is adopted, where the soil modulus depends on the confining effective stress, and the degree of mobilized strain. The friction hardening law involves the hyperbolic model parameters, with the failure ratio (R_f) set as 0.9.

The key soil parameters are E_0 and ϕ , which represent the initial Young's modulus (also the unloading/reloading modulus) at reference stress level of 100 kPa, and the peak friction angle, respectively. In the probabilistic analyses, E_0 and ϕ follow lognormal distributions, and their mean values for each soil layer are shown in Table 2, also following the values adopted in Pickles et al. (2006). The coefficients of variation (CV) for E_0 and ϕ are taken as 0.4 and 0.15, respectively, in the prior distributions, which are consistent with the suggestions by Phoon and Kulhawy (1999). In an ideal situation, site-specific CV for E_0 and ϕ may be adopted if there is sufficient experimental data of the parameters.

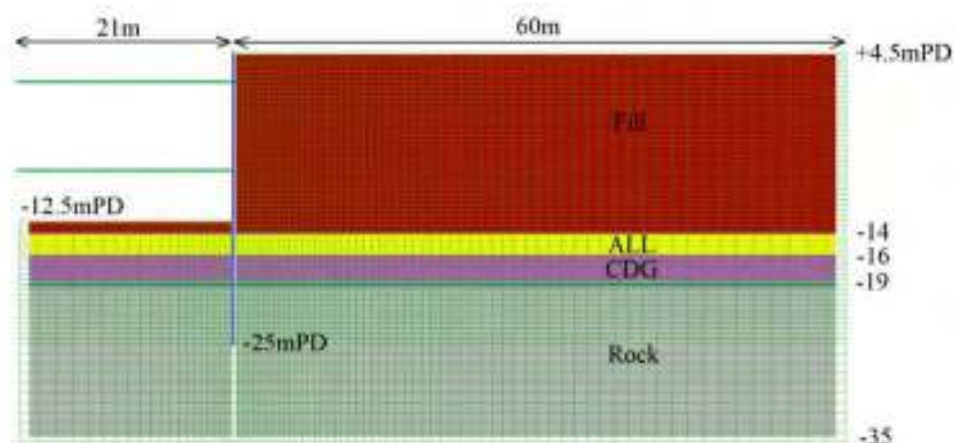


Figure 3: FLAC model of the analyzed cross-section at construction Stage 4

Table 2: Input parameters for FLAC Chsoil model

	γ (kN/m ³)	Mean E_0 (MPa)	CV: E_0	Mean ϕ	CV: ϕ
Fill	19	128	0.4	34°	0.15
Alluvium (ALL)	19	192	0.4	34°	0.15
Completely decomposed granite (CDG)	19.5	240	0.4	37°	0.15

In the FLAC model, the 18.5-m thick fill layer is further divided into upper and lower sublayers, in order to introduce more potential variations to the model parameters and hence predictions. Subsequently, 500 sets/combinations of E_0 and ϕ profiles are generated by Latin Hypercube Sampling, which can improve the coverage of the samples, and the 4 construction stages are simulated by FLAC. A FISH function (a programming language embedded in FLAC) is compiled to process the 500 cases automatically. Prediction intervals for wall deflections and settlements, defined as mean response (best estimate) plus and minus 1.5 standard deviation, can be evaluated based on the 500 simulations, and are shown in the later sections (Figures 7 and 8). These correspond to approximately 87% confidence that the actual excavation response lies in the prediction interval.

3.4 Principal component analysis and surrogate modelling on wall deflection and settlement

Although the number of measurement data points (i.e. 63) is large, they are highly correlated, which means adjacent measurements have similar deflection/settlement variations. In this case, PCA can be applied to extract features that capture most of the response variations. The excavation response can be decomposed into a product sum of the features with their respective principal component (PC) values. The variance of the PC value indicates the importance of the feature to the response variance. Applying PCA to the 500 excavation responses reveals that 5 features (which are conceptually similar to mode shapes in dynamic analyses) can capture 99% of the response variance, with 3 features for wall deflection and 2 features for settlement. An illustration for wall deflection at Stage 4 is shown in Figure 4. The 1st feature resembles the actual deflection shape. A large magnitude of the 1st PC corresponds to large overall deflections for the entire wall. The 2nd PC controls the localized variations in the wall deflection. The upper section of the 2nd feature is negative while the lower section is positive. A large magnitude for the 2nd PC leads to a bulging wall, with small deflections in the upper section and large deflections at lower sections. The 3rd feature represents localized variations that changes more abruptly than the 2nd feature along the depth.

The PCs of wall deflection and settlement are then related to the soil properties through a surrogate model. The surrogate model adopted in this study is called polynomial chaos expansion (PCE). For example, the PCE for the 1st PC of wall deflection is given by:

$$PC_1 = a_0 + a_1 E_{0,fill (upper)} + \dots + a_4 E_{0,CDG} + a_5 \phi_{fill (upper)} + \dots + a_8 \phi_{CDG} + a_{11} E_{0,fill (upper)}^2 + a_{12} E_{0,fill (upper)} E_{0,fill (lower)} + \dots + a_{78} \phi_{ALL} \phi_{CDG} + a_{88} \phi_{CDG}^2 \quad (2)$$

where a_0 is the bias term, $a_1 \dots a_8$ are the first order coefficients, and $a_{11} \dots a_{88}$ are the second order coefficients that capture the interaction effects between the soil properties. The coefficients can be determined by a regression analysis using the 500 sets of soil properties and the 1st PC of wall deflection from the analyses. In total, 20 PCE are built, since there are 4 construction stages and 5 PCs. If all the measurement data (individual inclinometer readings at 0.5m intervals) is used directly to build the PCE, a total of $4 \times 63 = 252$ PCE is necessary, and the surrogate model would be too complex for successful Bayesian updating. Finally, to evaluate the accuracy of the PCE and to avoid overfitting, the cross-validation coefficient Q^2 is used (Blatman and Sudret 2010). Q^2 is based on leave-one-out cross-validation, and does not always increase with number of predicting variables, thereby avoiding overfitting of surrogate model. In Eq. (2), only the coefficients that increase Q^2 are kept. As a result, $Q^2 > 0.975$ for all the cases analyzed in this study.

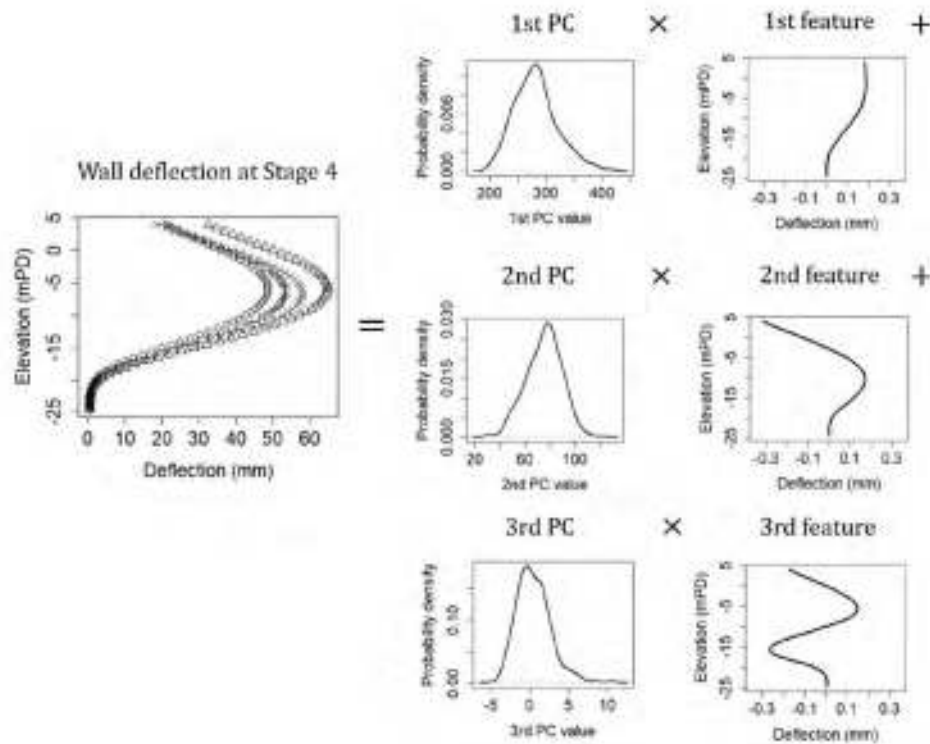


Figure 4: Illustration of principal component analysis of wall deflection at Stage 4

3.5 Sensitivity analysis

After formulating the surrogate model, sensitivity analysis can be performed to quantify the influence of each soil parameter to the variance of the overall system response. This study adopts the Sobol' sensitivity index (Sobol' 2001) as the quantitative indicator, which is a global index that measures the sensitivity across the entire parameter space. If a soil parameter is associated with a higher Sobol' index, it contributes more to the variance of the response. For example, results of the Sobol' index analyses for wall deflection are presented in Figure 5, which shows that ϕ of the upper fill layer is the most influential parameter across all stages. Meanwhile, ϕ of the lower fill layer becomes more influential at later stages, as the shear strength as depth becomes mobilized at those stages. In general, the influence of E_0 diminishes as the excavation progresses with higher strength mobilization. In addition, parameters in the alluvium and CDG layers are of negligible importance, due to the relatively small thickness. Through this sensitivity analysis, one can become better informed of the relative importance of various parameters, and which parameters will be refined more effectively through Bayesian updating at different individual stages.

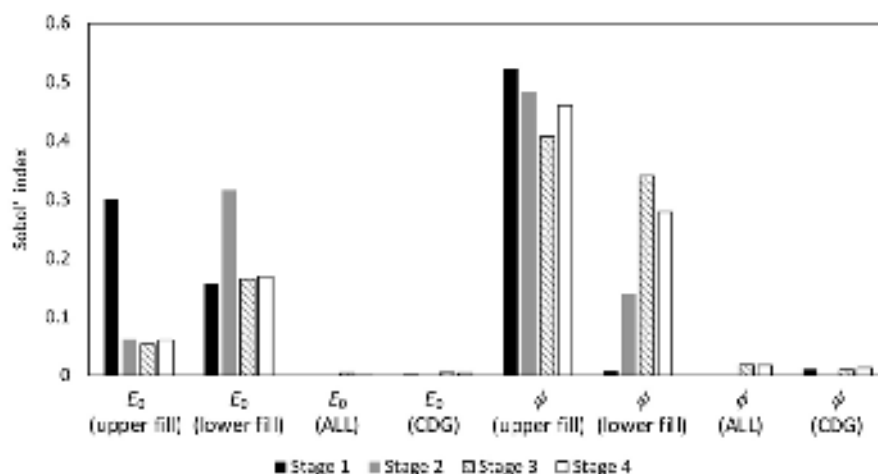


Figure 5: Sobol' sensitivity index for wall deflection at each construction Stage

3.6 Results of Bayesian updating

During the construction stage, when wall deflections and settlements are measured, the information can be used to obtain posterior distributions of soil parameters through Bayesian updating. Tables 3 and 4 show the posterior mean and standard deviation of the parameters after updating at each stage by the MCMC algorithm, while the prior and posterior distributions for fill properties are shown graphically in Figure 6. The standard deviations (uncertainty) of the soil properties keep decreasing through successive updating, as the response and information from multiple stages are utilized. The reductions in standard deviation are more prominent in the fill layer, because it is the most influential to the wall deflection, as revealed by the sensitivity analysis (Figure 5). The updated E_0 in the fill layer is much lower than the prior specification. Also, the updated E_0 and ϕ values are substantially different in the upper and lower fill layers, which justifies dividing the fill material into two sublayers in the analyses.

Table 3: Mean and standard deviation (SD) of E_0 after Bayesian updating at each construction stage

	Upper Fill		Lower Fill		Alluvium		CDG	
	Mean	SD	Mean	SD	Mean	SD	Mean	SD
Prior	128	51	128	51	192	77	240	96
Stage 1	92	21	58	12	200	55	258	73
Stage 2	89	16	55	8	182	41	232	51
Stage 3	90	14	58	7	189	30	230	41

Table 4: Mean and standard deviation (SD) of ϕ after Bayesian updating at each construction stage

ϕ (°)	Upper Fill		Lower Fill		Alluvium		CDG	
	Mean	SD	Mean	SD	Mean	SD	Mean	SD
Prior	34	5.1	34	5.1	37	5.6	37	5.6
Stage 1	34.7	2.5	33.9	4.5	34.4	3.4	39.2	4.4
Stage 2	35.7	1.8	33.7	3.4	33.6	2.6	40.4	3.9
Stage 3	35.6	1.5	38.7	3.1	33.7	2.3	41.5	3.6

Figures 7 and 8 compare the prediction intervals of wall deflections and settlements before and after Bayesian updating. The updated prediction intervals at a certain stage are based on the updated soil parameters from the previous stage. The dark grey region corresponds to mean plus and minus 1 standard deviation, while the light grey region is obtained by the best estimates plus and minus 1.5 standard deviation. The updated prediction interval in Stage 2 shifts outward (for deflection) and downward (for settlement) towards the actual response, due to the reduced E_0 values after updating. For Stages 3 and 4, the measured wall and ground responses are generally encapsulated by the prediction intervals, and the Bayesian process leads to reduced uncertainty in geotechnical parameters and narrower prediction

intervals of the response, compared to probabilistic analyses without the updating process. In fact, it is possible to improve the accuracy of predictions by incorporating more frequent updating stages into the proposed framework, since site measurements are retrieved and reported almost daily for deep excavations in major infrastructure projects.

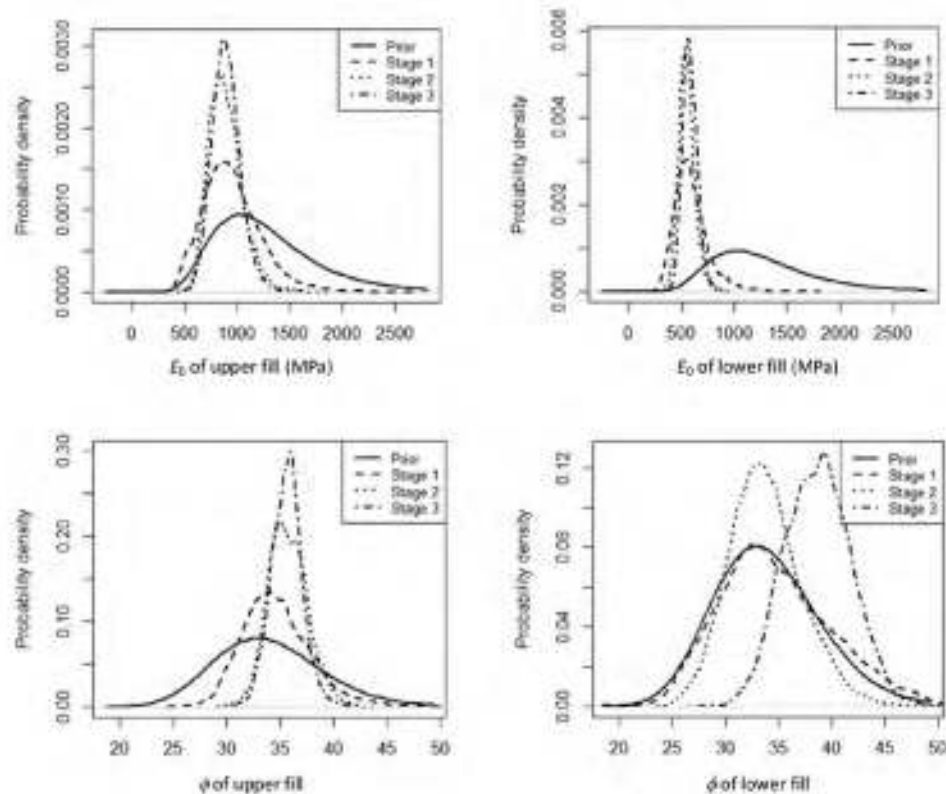


Figure 6: Prior and posterior distributions for fill properties

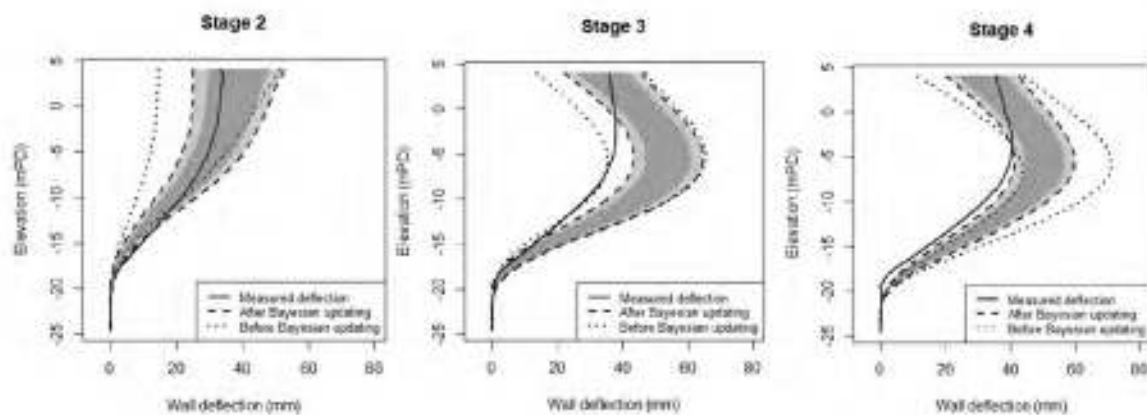


Figure 7: Prediction intervals of wall deflections before and after Bayesian updating

4 DISCUSSIONS

Many probabilistic approaches require multiple sets of numerical analyses. In this study, 500 numerical analyses were performed to investigate the influence of different combinations of soil properties, but these analyses can be easily automated by commercially available software such as FLAC. Therefore, the role of engineers rests on defining the best estimates and uncertainty (standard deviation) of relevant geotechnical parameters, through carefully designed ground investigation. The corresponding simulations can then be automatically performed without a manual trial-and-error calibration process. Also, all the finite difference or finite element analyses of the multi-staged response will be performed

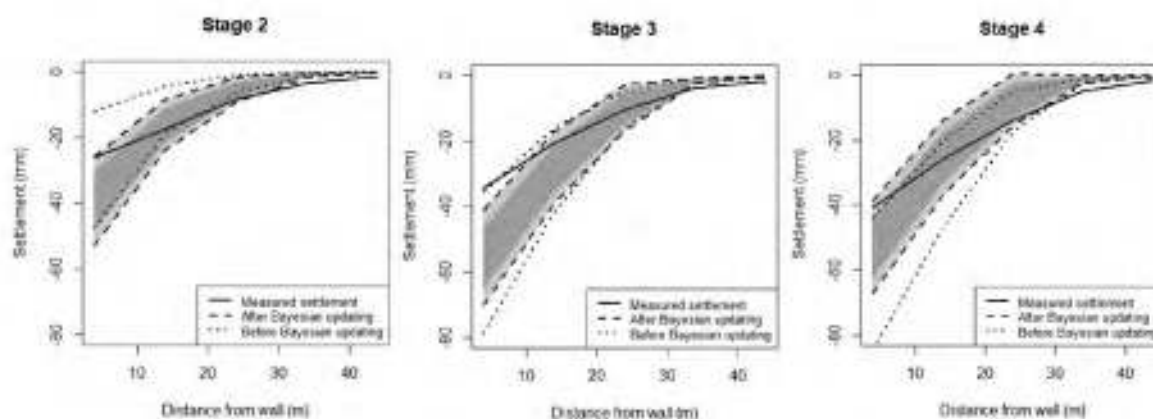


Figure 8: Prediction intervals of ground settlements before and after Bayesian updating

during the design stage, prior to commencement of construction. When monitoring data becomes available during construction, the updating algorithm (MCMC) and surrogate model analyses for refined predictions can be completed in minutes using typical desktop computers. Meanwhile, monitoring data should be measured frequently from multiple sources/ sensors, with a wide coverage across the construction domain to enhance the effectiveness of the approach.

The proposed updating approach and probabilistic assessments provide information on the likely system response at different levels of confidence, where prediction intervals provide quantitative indicators on the probability of system behavior exceeding certain limits. They may be adopted as rational input to the establishment of alert-alarm-action (AAA) levels, as the intervals represent the potential ranges of system response considering natural variability in the subsurface materials. It would be a cause for concern if field measurements exceed the intervals, as this implies that some elements of uncertainty may not have been properly accounted for, or the actual construction sequence may differ from the modelling conditions. The proposed approach also contributes to several key elements of the observational method as outlined by Spross & Johansson (2017), including the assessment on the range of possible behavior, and probability that the actual behavior will be within acceptable limits, and also the incorporation of monitoring plan with frequent measurements to allow timely review of the system response and design assumptions.

The proposed approach may be further improved to incorporate spatial variability of geotechnical parameters, even within the same stratum. This would involve advanced probabilistic techniques such as random field modelling, to include more localized variations in soil properties. Such modifications will likely improve the accuracy of the approach, and will be the focus of future research.

5 CONCLUSIONS

This paper presents an adaptive updating approach that utilizes site monitoring data, such as wall displacements and ground settlement measurements for a deep excavation, to update the knowledge on geotechnical parameters and the associated uncertainty. The approach allows continuous refinements on the response predictions for subsequent construction stages, providing probabilistic estimates at different levels of confidence to support decision making. Although it requires a larger number of numerical analyses at the design and planning stages of the project, these can be easily automated in many commercially available analysis software.

ACKNOWLEDGEMENTS

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Connected Data – A digital journey in geotechnical engineering

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ABSTRACT

The McKinsey digitization index has listed the construction industry as one of the most undigitized industries. Although people have been talking about digitization for years and how it helps different industries to grow, it still feels digital is far away from construction industry.

This paper will go through a digital transformation journey and its implications on geotechnical engineering. It will cover a wide range of technologies including BIM, Apps, IoT technology and AI analytics, and their applications throughout the whole construction cycle, from design to delivery. By integrating different digital solutions into construction procedures, it streamlines construction processes, as well as improving productivity and efficiency.

Key Words

Digital Transformation, BIM, IoT, AI, Digital Twin, Geotechnical Engineering

1 INTRODUCTION

Compared with other industries such as finance or retail, construction industry always builds in a dynamic environment that is never the same, which makes it extremely hard to capture and digitize spatial and non-structural data. However, advancement in digital technology is slowly changing the situation, and gradually redefining how construction can be done.

The need for digital transformation in the construction industry is growing. The transformation is about connected data, linking and mapping different procedures, such that it could be streamlined, automated or even be able to generate new business models. Building a common data environment that centralizes all digital data captured at different stages of construction projects will help us take this leap in digital transformation, and create new opportunities for the construction industry. The following section will cover the digital journey in Gammon Construction.

2 CONNECTED DATA IN GEOTECH ENGINEERING

The connected data stream starts from the ground investigation, then design stage all the way down to construction and data analytics. Different technologies are adopted at different stages to capture data, and create a connected data stream to link up all procedures. Processes are streamlined, automated, and sometimes even create new opportunities or procedures. The following sections will discuss the benefit of connected data, and how connected data could improve construction works with practical innovation examples in Gammon Construction.

2.1. Borehole GIS

Data is captured even before any construction design work is started. Traditionally GI data are stored and exchanged using borehole PDF. PDF is a straightforward way for people visualize, but it is not easy for computer to interpret it directly. In Gammon, a Borehole GIS system is set up to store GI in both digital AGS format and PDF borelog format.

This allows people to instantly transform GI into other format e.g. SPT plot, or run other numerical analysis e.g. preliminary stratum layer determination. Furthermore, with these digital data easily accessible on cloud, designer can instantly transform these raw numerical data into 3D model within seconds, using parametric modeling approach.



Figure 1: Borehole GIS system



Figure 2: Parametric 3D modeling directly from AGS

Designers can now easily inspect the relationship between adjacent boreholes, and visualize corbel area or fault zone in 3D. This system speeds up the GI data exchange and processing speed, getting the required information instantly. This also gives designers greater visibility to the ground condition, hence eliminating part of the risk in geological projects.

2.2. Design in BIM

After getting GI information from the cloud based Borehole GIS system, designers can interpret this information, and append design information on top of 3D GI data. Using parametric BIM design procedure developed in Gammon, designers connect their design calculation directly to BIM through dynamo script or other plugins.

With this parametric approach, designers could generate a BIM model within a relatively short period of time and at lower cost. Different parties can add their information into the model, and collaborate to eliminate interfacing risks.

Everyone knows BIM is important, but bringing the cost down is the only way to make it practical and useful. The key is to control the flow of digital spatial data properly at different stage of construction.

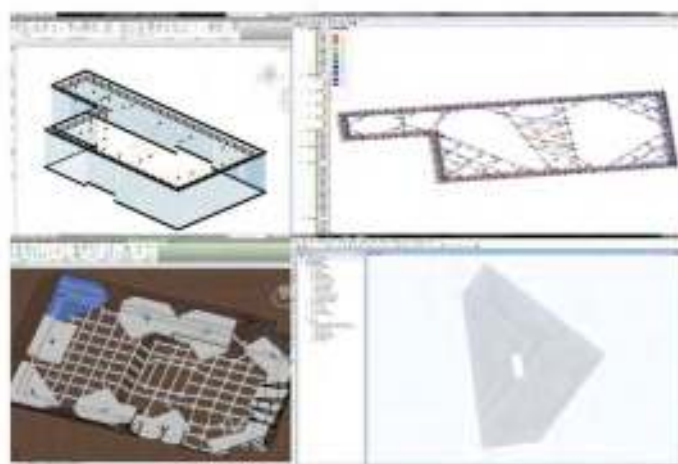


Figure 3: Parametric BIM in Design



Figure 4: BIM collaboration between design and tender estimating

With all the data exchange workflows in place, it is possible for Gammon to create BIM model at a relatively low price, and prepare a model for every single tender. Tender estimating and quantity take off can be done directly from BIM model, extracting empty bored length, founding level, excavation volume calculation etc.

2.3 Foundation E-Data

BIM model is then passed to construction phase for 4D planning. Planning information is added on top of 3D model data to visualize construction sequence, inspecting conflict in construction plant logistic, haul road design etc. Gammon also set up its Foundation E-data system to capture and monitor site progress. With mobile apps and other user interface developed, operation teams can easily update their information to Gammon Cloud, a single source of truth for site progress. The as-built progress is



Figure 5: Foundation E-Data Dashboard



Figure 6: 4D BIM for planning on top of design BIM data

compared against planned progress prepared in 4D, triggering alert if progress does not go as planned. The project management team can trace the full record of all construction progress, without duplication or lost record.

Different teams can easily see the progress of other teams, hence improving collaboration and avoid confusion. Different reports and dashboards are directly linked to Foundation E-data system, hence PM can access information anytime and anywhere.

2.4. IoT for labor and plant tracking

Other than planning and progress data, Gammon also collects construction plant efficiency data and labor productivity data with IoT technology.

Different sensors are installed on various types of equipment, collecting data regarding operation hour, idling status, location, fuel consumption, rotation speed, temperature, pressure etc. This information gives projects more control on different equipment, and assists in understanding how well operation teams are utilizing project resource.

Beside plant and equipment tracking, labor is another important resource for construction projects. Understanding the behavior and productivity of workers is critical for construction project management.

Bluetooth beacons or CCTV are installed on site to monitor labor count. Workers' attendance and location are tracked, and correlate with progress data to calculate the productivity of workers.

All data including design data, planning data, progress data, labor, plant and equipment are centralized at ACE platform, a cloud based dashboard platform, for project teams to visualize, collaborate and manage project progress, achieving digital project management.



Figure 7: Sensor of different construction equipment and management dashboard

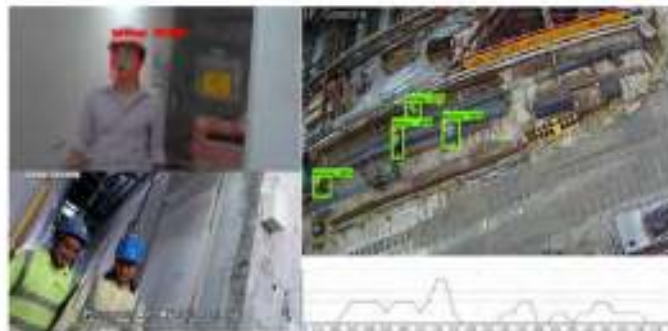


Figure 8: BLE and image recognition for labor tracking

2.5. Productivity analysis and Risk Evaluation

Data generated from construction life cycle, starting from GI, design, planning, procurement, down to construction as-built record are captured in a common data environment. This allows joining multiple data sources together, and helps to discover more insight from a holistic view, e.g. combining progress data with plant drilling sensor data and GI data to investigate the performance and productivity of equipment under different stratum, or the variation/risk in different area.

These data also support more advanced machine learning analysis to determine program risk based on historical piling data and variation. People can use data from the past to predict the future.

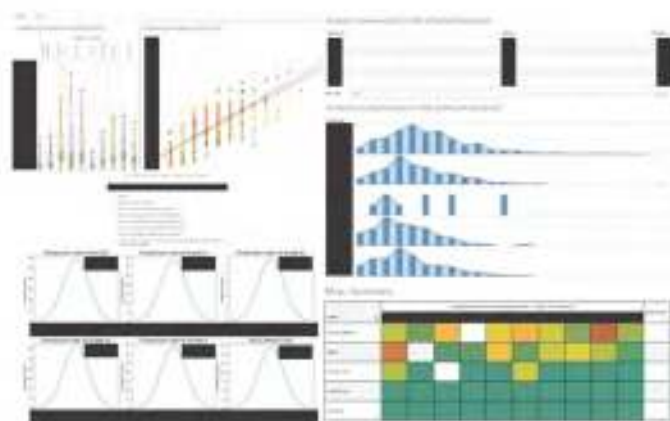


Figure 9: Big Data analytics for foundation piling works

2.6. Digital Twin

All connected data stored in a common data environment are eventually streamed to a 4D visualization engine, and reconstruct a digital twin with parametric approach. Information including piling progress and plant location will be streamed to a digital twin and presented in a 4D graphical way.

This model will update automatically as data changes. Project team and top management could monitor and communicate site progress in a 3D environment, which is easier to understand and communicate.

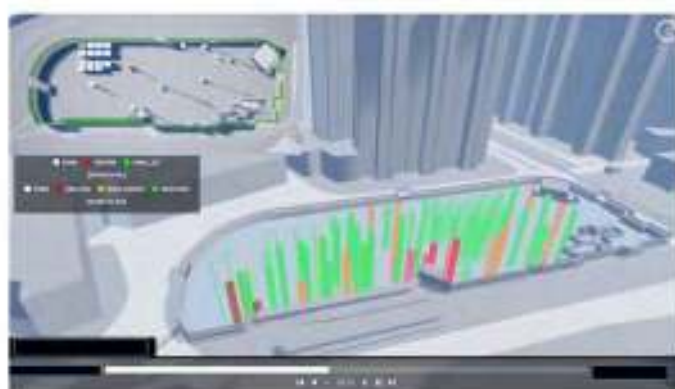


Figure 10: Digital Twin Streaming actual site progress

3 CONCLUSION

This paper briefly describe an example of connected data, and how it can be applied in geotechnical engineering. Data stream should start from the very beginning of the construction life cycle, going from design and planning down to construction. Information should be transferred to subsequent parties

seamlessly, such that each party can use the information directly, and append new information into the data stream for the next party.

Individual innovation is eye catching, and can bring benefits to the industry, but most benefit will be generated by integrated data. In some cases, connected data can create new processes, such as advanced analytics mentioned above, or even new business opportunities and transform how construction works can be done.

It is not always easy to do data exchange across multiple parties. However, this is the ultimate problem that the construction industry needs to solve. If it wants to go through digital transformation and generate more value for the society.

4 ACKNOWLEDGEMENTS

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The Use of Building Information Modelling

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ABSTRACT

Building Information modelling (BIM) is now an integral part of Civil Engineering Projects through concept, design, construction and maintenance stages. The detail and input depend upon the project type and specification. The uses have been valuable for clash detection, such as utility installation, which is typically located below ground, out of view and within congested space prone to impacts and delicate interfaces with other construction works. Despite the advances, challenges exist for compatibility and transfer of data between different software types and application and a consistent communication between the many specialist parties using BIM. This paper provides an overview of the history, definition and understanding of BIM and its application to civil engineering. A summary of international published data with particular focus on its use for utility design and construction for infrastructure projects is provided. For geotechnical engineering a recurring problem can be the imbalance between BIM presentations which may detract from the more mundane but vital input of accurate factual data.

1 DEFINITION OF BUILDING INFORMATION MODELLING

BIM definitions depend upon the application, project and specification requirements, due to the changing nature of BIM application there has been difficulty adopting standard definitions, as referenced below:

- ‘creation and use of coordinated, computable information about a building project in design and construction’ (Van Nederveen et al, 1992).
- “digital representation of physical and functional characteristics of a facility” (Down-to-Earth technologies, 2014).
- ‘a reliable basis for decisions during a project life-cycle from earliest conception to completion, either demolition, as built or maintenance stages of a facility’ (Wikipedia, 2014).
- ‘application representing characteristics through planning, design, construction, operation and maintenance stages for different physical infrastructures, such as water, wastewater, electricity, gas, refuse and communication utilities to roads, bridges and ports, from houses, apartments, schools and shops to offices, factories, warehouses’ (HKBIM, 2014 & Smith, 2007).

BIM became a popular reference by software producers, initiated by Autodesk as a standard term for modelling multiple views on buildings (Down-to-Earth technologies, 2014) and for ‘digital representation’ by Graphisoft (Virtual Buildings), Bentley Systems (Integrated Project Models) and Autodesk (Vectorworks).

2 WORKFLOW

Choices are made for data exchange during design and construction. Given the variety of software application, interfaces and demand for rapid processing, a challenge is to establish a Computer Aided Design (CAD) platform with a team of operators with appropriate experience to ensure the BIM workflow is carried out effectively. This is typically set up within a Specified BIM Implementation Plan.

A part of the exchange process is capturing site data and transfer. Methods to accurately map underground utilities is a typical example and can be carried using surface geophysics, survey, data management and non-destructive excavation applications. In order to achieve high levels of accuracy, GPS data communication techniques such as Real Time Kinematics (RTK) can be used; in the absence of ambient radio signals, can achieve accuracy to a few centimeters (Ehsani et al. 2004). An RTK rover antenna together with a radio signal receiver, placed on the excavation equipment, can continuously pass data to the base station. The exact position of the RTK rover antenna can then be ascertained for input to a visualization system. This can include software such as AutoCAD 360, Turbo View Pro (IMSI design), BIMx (Graphisoft), Genius Scan (Grizzly Labs), Photo Measures (Big Blue Pixel) and MagicPlan (Sensopia), Ward et al, 2014.

The BIM standards, applicable to Hong Kong, are generally categorized in a Project Execution Plan (PEP). The aims of the modelling method, level of detail and style are summarized in Table 1 (HKBIM, 2014).

Table 1: BIM Categorization and Aims

	Standards Category	Aim
i	PEP	Project management and execution definition for the strategy, collaboration, production and data segregation;
ii	Modelling Methodology	Model development, including the efficient use and consistency of the BIM data applicable to each discipline;
iii	Level of Detail	Specify the intended graphical scale, level of detail (architectural and structural) models for each project stage, including the design (concept, preliminary and detailed), approval, construction and as-built;
iv	Presentation style and data organisation	Standardisation of appearance, style, size, properties, categories, units and measurement, data structure and nomenclature.

The initial BIM set up, is typically initiated by the client prior to receiving appropriate data from the design consultant and the contractor at later stages. See Figure 1 for the process.

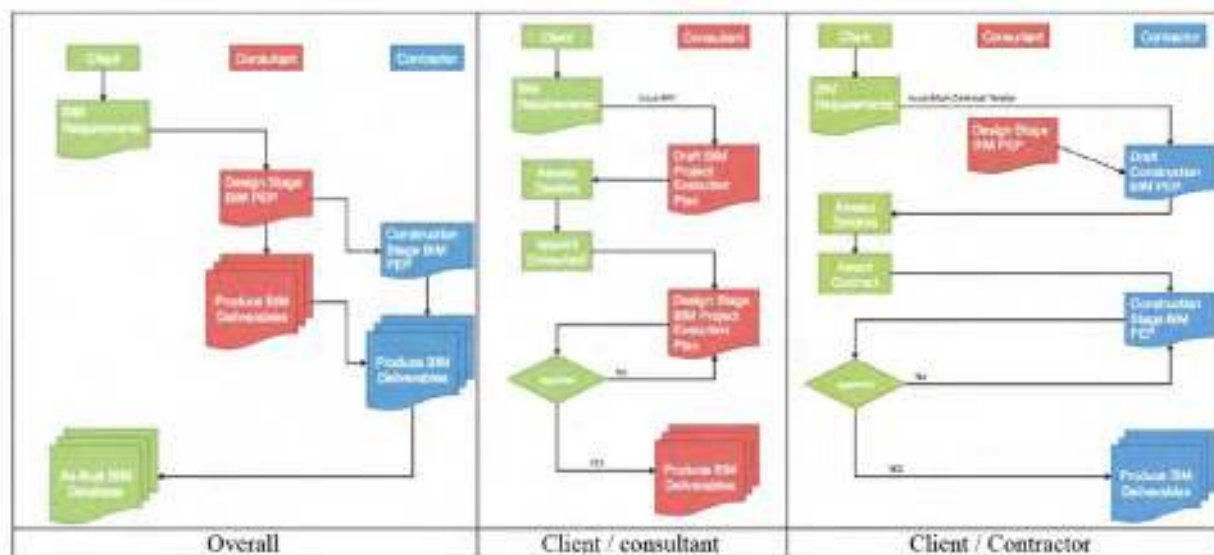


Figure 1: BIM work flows between different parties (HKBIM, 2014)

During construction stage the BIM PEP should be passed to the contractors for review and consideration for amendment and update of the project deliverables. The software and file exchange models, such as DWG, DGN, DWF, PDF and IFC files need agreement at each stage (HKBIM, 2014).

An example of the application of BIM standard is set out within the Hong Kong Mass Transit Railway Corporation Limited (MTRCL) Specifications. A typical standardised drawing following the MTRCL CADD Manual is required with definitions provided for the Modeling and Level of Detail covering the full extent of the civil, and architectural works. The building services and system-wide Electrical and

Mechanical (E&M) input is also needed with the aim of providing effective transfer between the client and different parties involved in the project. The requirements are typically provided for the naming conventions, families, file exchange protocol and symbols and annotations, typically initiated by a nominated BIM originator, who then typically sets up the following requirements:

- BIM Authoring Application Version;
- Model Geometry (co-ordinates, setting out, levels & units);
- Model Division (model demarcation, central/local file setting & workset setting);
- Model Project Browser Set-up;
- Model File and Revit Family Naming Convention;
- Model Maintenance and Quality Control Procedures;
- Revit Family Library Descriptions;
- Clash Analysis System;
- BIM Deliverables Descriptions (native files, review files, clash analysis reports structure & file/folder structure).

3 INTERNATIONAL CASE HISTORY EXAMPLES

3.1 Port Mann Highway, Canada

BIM was used as a management tool during the design and construction processes for the Port Mann Highway Interchange project (NCEI, 2013), near the Port Mann Bridge, Highway 1, Vancouver Canada (See Figure 2).

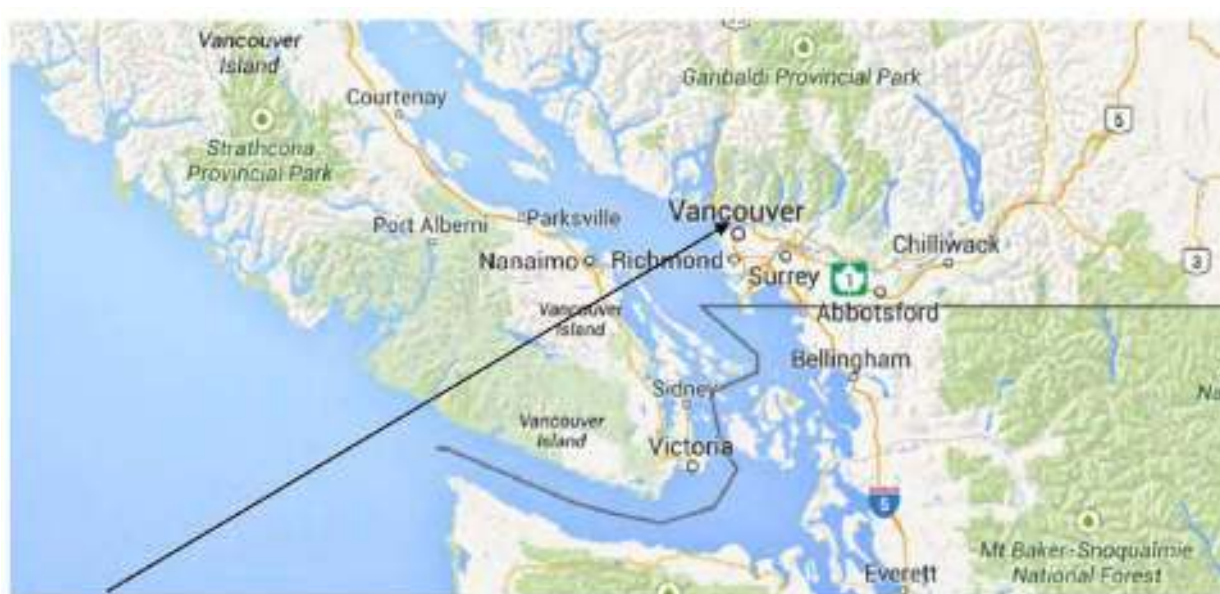


Figure 2: Port Highway Interchange, Vancouver, Canada (Google Earth, 2011)

The initial stage involved the use of Autocad software to create a virtual 3D corridor, providing the highway grading, layout and geometry for the 37 kilometre alignment; See Figures 3 and 4.

The initial 3D visualization allowed presentation of the utility interfaces (drainage culvert, power line and sewers) allowing adjustment of the alignment or utility realignment to limit interference. This was updated as more detail became available (NCEI, 2013). Once the alignments were set the construction sequences could then be determined. The main challenge for the use of BIM was the data exchange between different parties, software compatibility and the provision of an appropriate level of detail for the type of works adopted.

The alignment was placed over ground terrain models determined by topographic detail provided from on-site laser scanners. This was particularly useful in locations in close proximity, to the alignment corridor.

The coastal environment was highly changeable, prone to storm surges and tidal influences. It was therefore imperative that the alignment from land to sea was located with minimal coastal impact, as far as possible, and had sufficient coastal protection. The initial route selection therefore involved environmental impact study and preliminary design carried out by placing available digital data, such as wave impact and coastal protection, into a database (ArcGIS). This was used for storage, organization and management of the data. The various pipeline alignments were then considered using visual presentations to show interaction with the landfall facilities (see Figures 9 and 10) and highlight the main environmental impacts.



Figure 9: 3D BIM model of the pipeline compound



Figure 10: Preliminary model

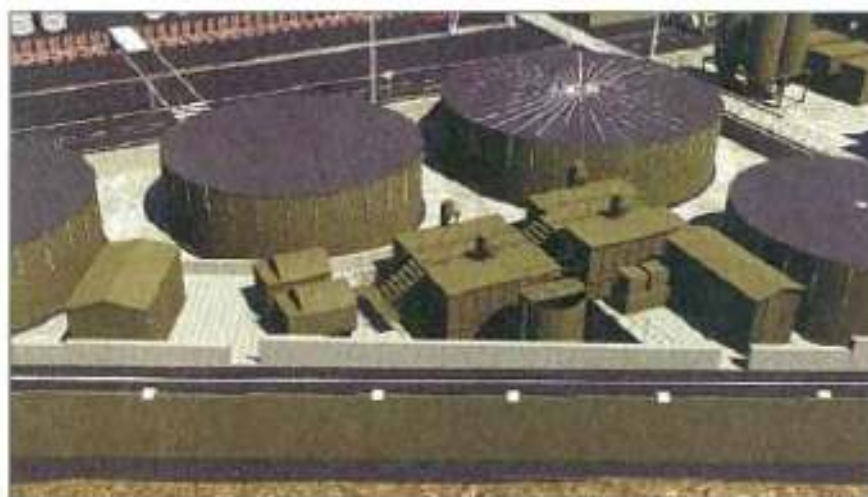


Figure 11: Surface Water Treatment plant model

During the detailed design one of the considerations was the plant, equipment and operations design within the compound. This was presented as a 3D visualization to all parties providing rapid planning and technical collaboration between all relevant parties (see Figure 11). As the pipework data was provided during the later program stages this allowed rapid clash detection assessment between the construction and design teams allowing alignment adjustment. The process was continued through to construction for the drainage layout design in congested locations allowing rapid layout optimization (Ward et al, 2014).

3.3 Tottenham Court Road Station, London, UK

Tottenham Court Station is located in central London at the intersection of the Central and Northern Lines of the London underground transport system. The station upgrading works commenced in 2007 and completed 2016, costing approximately 1 billion pound sterling (TfL, 2013). Refer to Figure 13 for a visual presentation of the site (Li et al, 2014). BIM was used in the design and construction process for this project to:

- Design the constraints imposed by the limited space available for the site works;
- Provide visualization to assist interfacing between the different design disciplines (see Figure 21);
- Integrate the design model with the site topography to ensure compatibility with actual site conditions;
- Visualise the model at different construction stages.

Upon completion of stage of the works program a visual image was produced to record the site progress at that stage used to represent the progress with time (4D views) of the works; see Figures 12 to 15, Li et al, 2014. The updated records allowed a 3 to 4 month look ahead with associated construction program updates.

In addition to the use for planning and programming BIM was also provided overlays for other specialist input requirements and updates. Part of this process was enhanced by regular survey data updates from Ordnance Surveys and laser imaging detection and ranging (Lidar) data. The 3D information included the topographic map, utility services, the site boundary, temporary works, station architectural details and the ground model.



Figure 12: General sketch presentation, Tottenham court road



Figure 13: 3D construction model



Figure 14: Site photograph of the works (11/12)



Figure 15: Visual image of the works (11/12)

4 GEOTECHNICAL CONSIDERATIONS

BIM has been an essential tool to present ground models updated as increased data becomes available. An example of the presentation of depth to 'rock-head' for an arbitrary location is presented in Figure 16.

For the case represented in Figure 17 this provided a clear indication of the rock-head for a foundation design allowing rapid and clear presentation of the variation in depth of foundation installation with associated cost and time for all non-technical stakeholders. Notwithstanding the presentation was oversimplified requiring an interface with a Geographical Information System (GIS) to ensure all

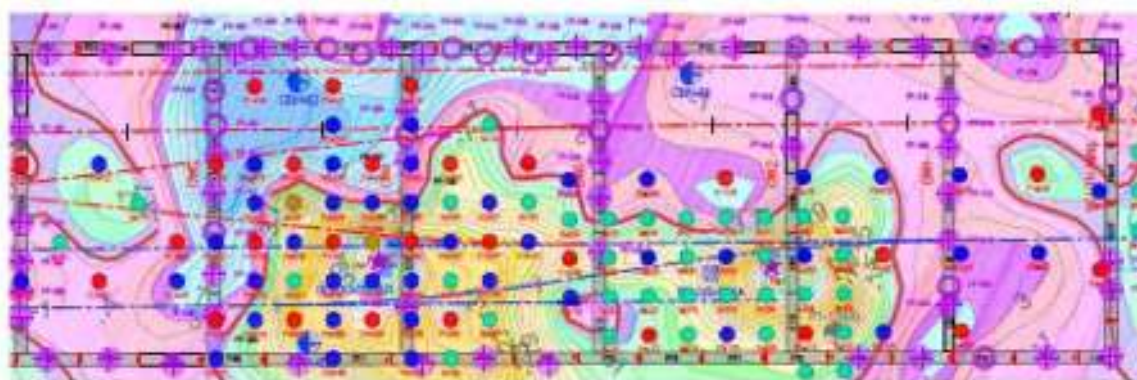


Figure 16: 'Rock-head' contours interpreted from probe data (depth to 'rockhead' is presented by colour)

relevant data could be input and used to effectively present all considerations. As a result the risks associated with an oversimplified presentation included:

- arbitrary 'rock-head' definition
- omission of vital data, such as complementary boreholes and
- lack of consideration of major risks, including the presence of weak zones beneath the initial encounter of rock.

CONCLUSION

The use of BIM as a project management tool in planning and visualizing construction projects is standard requirement used throughout a civil engineering project cycle. Attention is needed in consistent use of software and transfer data to ensure its efficient use through each project stage. Its use can lead to significant cost and time savings. For sub-surface infrastructure attention is needed to ensure there is not an imbalance between the effort required for a clear presentation without due attention given to the accuracy and relevance for the data presented.

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Any opinions expressed in this paper are those solely of the authors and not of any other party.

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The Use of Underwater Cone Penetration Testing in Hong Kong

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ABSTRACT

The Cone Penetration Test (CPT) has been used for decades to determine geotechnical parameters for the soft to firm superficial deposits typically overlying the sea bed profile in the Hong Kong Special Administrative Region (HKSAR). More recently CPTs are required to fulfill contract requirements to ensure the seabed deposits have gained sufficient strength to place reclamations for future land use. An example of this has included the use of deep cement mixing (DCM) ground improvement methods to improve weak ground located beneath the 650 Hectare Chek Lap Kok Airport third runway extension, which required the use of CPTs to verify the success of the improvement. Due to the time and cost constraints to carry out CPT tests over extensive and congested areas, a reaction frame placed on the sea-bed level from a crane extended from a static marine barge is typically used. This paper provides an overview of the GEOMIL CPT system used by Lam Geotechnics Limited as a relatively cheap and rapid method to advance the CPTs from the seabed. A brief outline of the history and specified requirements for the CPT requirements for DCM ground improvement is provided.

1 THE HISTORY OF THE CPT IN THE HONG KONG SPECIAL ADMINISTRATIVE REGION

The CPT was developed in the Netherlands in 1934 to determine the presence and density of sand layers within soft deltaic deposits and was introduced to Hong Kong Special Administrative Region (HKSAR) during the 1970's (Pyle et al, 1970). From the 1980s pore water pressure values were recorded using piezocones in the HKSAR through the use of porous elements with transducers fitted within the cone (Koutsoftas et al, 1987). Refer to Figure 1 for the Piezocone configuration (GCO, 1987).

CPTs driven from the sea-bed use a wheel drive system installed within a reaction frame to advance the rods attached to the CPT Cone. This was initially used to investigate the superficial deposits beneath the Chek Lap Kok Airport trial embankment during the 1980s (Plant et al, 1998). Refer to Figure 2 for a typical wheel drive configuration. Recently there has been advancement in the use of underwater CPTs through more accurate positioning techniques, real time recording and rapid digital reporting and transfer.

2 THE USE OF THE CPT

The CPT is a rod pushed into the ground recording the tip (cone) and shaft (friction sleeve) resistance. The rate of penetration is typically advanced at about 20 mm/s and inclination, pore-water pressure and loads recorded through cables and an inclinometer within the rod connected to recording devices at the

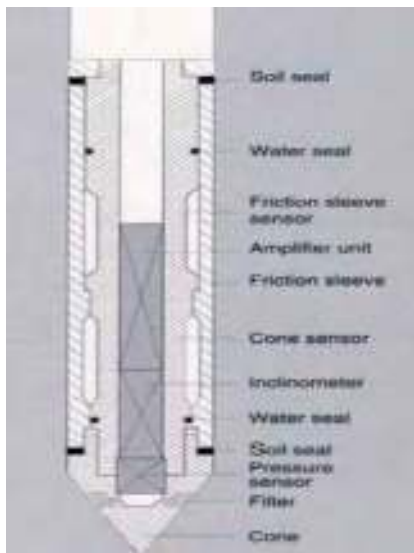


Figure 1: CPT Configuration



Figure 2: Wheel Drive System

surface. Regular, specified calibration checks of the drive and force-measuring systems are essential to ensure the results are representative. The CPT typically has a specified areas of 1000mm for the cone cross-section, with an apex angle of 60°, and a 15000 mm² friction sleeve area; refer to Figure 3, BSI 5930, 1999. The accuracy of the records are typically within $\pm 5\%$ of the actual value (GCO, 1987).

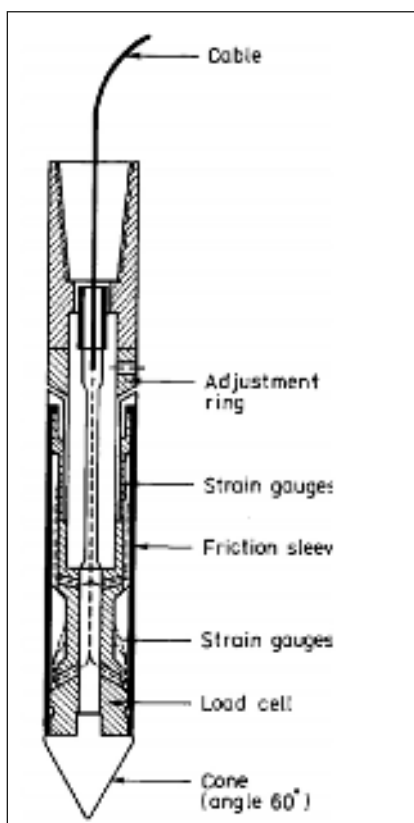


Figure 3: CPT Configuration

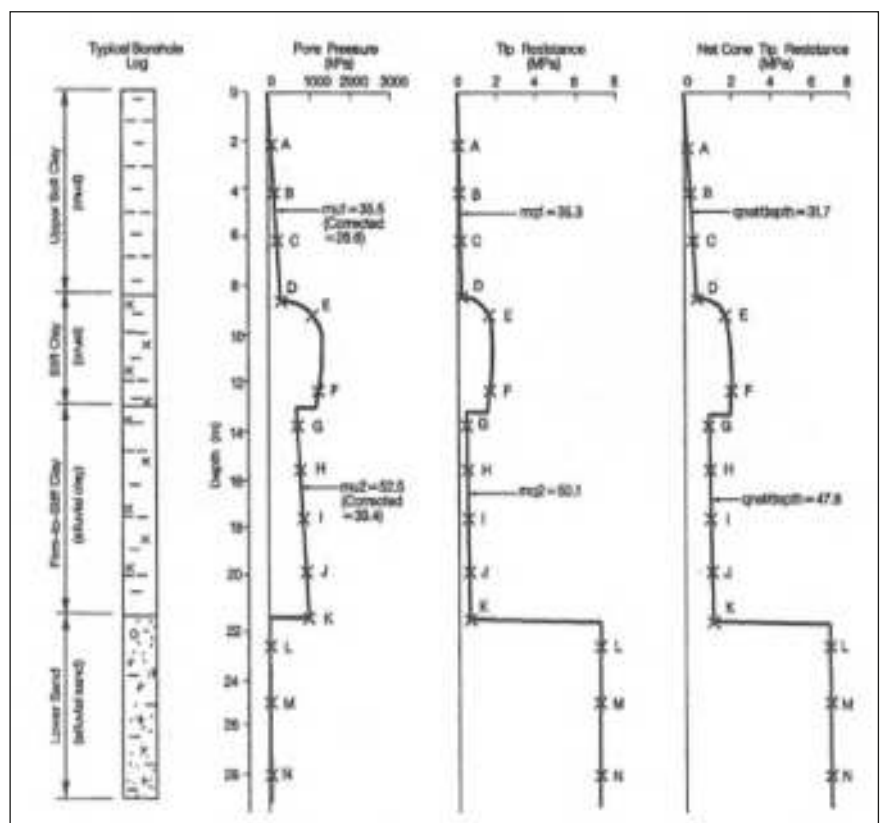


Figure 4: Idealised CPT correlation against borehole record

The advantages and disadvantages for the use of the CPT are described in Table 1.

Table 1: Advantages and disadvantages of the CPT systems

	Description
Advantages	• Provides a continuous soil profile, through interpretation of the friction sleeve and cone resistance ratio; pore water pressure records; • direct correlation with boreholes and vibro-cores; • relatively rapid and cheap investigation method compared with boring through soft to firm marine deposits, which require delicate sampling and testing techniques.
Disadvantages	• Restriction in loading, typically up to 6MPa, causing difficulties advancing the cone through dense sand, gravels, cobbles and rock with a weathering intensity below highly decomposed. • For CPTs advanced from the sea-bed the reaction frame requires sufficient load to ensure horizontal and secure placement on the seabed.

During the 1990s the CPTs were able to identify thin strata, of less than 10 cone diameters thickness (Campanella et al., 1982). This has since improved through more accurate and representative recording techniques during recent decades. Refer to Figure 4 for an example of an idealised CPT and borehole profile for records obtained during CPT advancement (Endicott, 1992).

3 THE USE OF THE CPT FOR NON – DREDGE RECLAMATIONS

During the last decade there has been a need to reclaim land in the HKSAR whilst fulfilling the environmental ordinance for non-dredging of the underlying potentially unstable soft marine deposits. The need for ground improvement has therefore been required; for the on-going Chek Lap Kok Airport 3rd runway reclamation the use of DCM installation has been adopted to fulfill this purpose. The generally accepted specification for DCM installation EN 14679, 2003. As stated “*the objective of the DCM is to improve the soil characteristics through an increase in shear strength characteristics, soil deformation properties and homogeneity of the DCM columns*”. Compliance with the design considerations to achieve these properties is therefore required.

As CPTs are able to provide a continuous soil profile, the “*investigation of the uniformity and indirect determination of mechanical properties*” of each DCM column is carried out using CPTs for unconfined Compressive Strengths (UCS) up to 4MPa. For the CLK 3rd runway ground improvement four overlapping DCM columns were typically installed to form a “cluster” for the ground improvement (See Figure 5). The cluster pattern and spacing was dependent upon the loading and intensity of ground improvement required, refer to Figures 6 for an indication of the cluster pattern located towards the north-east portion of the 3rd runway reclamation and Figure 7 for an example of a seawall configuration.

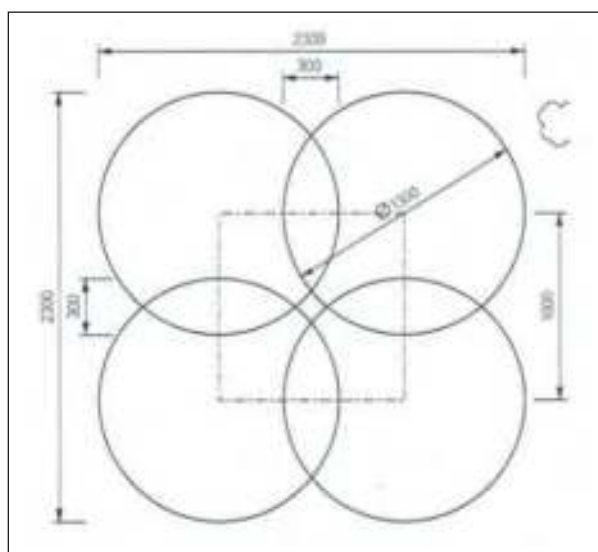


Figure 5: Typical configuration for a DCM “Cluster”

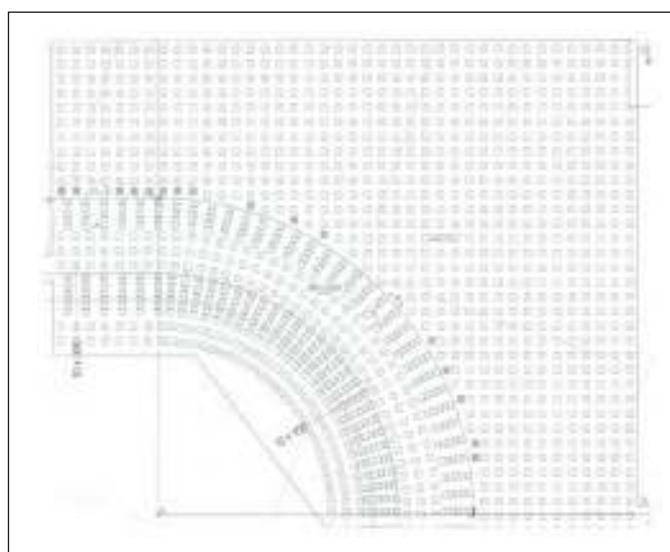


Figure 6: Example of the cluster patterns, NE 3rd runway

EXAMPLE

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$M = 27.700 \text{ kg} \quad 4500 + 4 \times 2400 + 5000 + 4 \times 1900 + 1000$

Width = 3000 mm Width (with/without skirt plate) = $2000 + 483 \times 2 = 2966 \text{ mm}$

To reduce deviation from the vertical the CPT rod is fitted with a dual wheel drive within the uppermost and lowest levels of the reaction frame to advance each segment of the CPT rod. Refer to Figure 11. The Reaction Frame can also be placed within the vicinity of the static barge provided that the CPT rods can be attached from a suitable vessel located at sea-level. Refer to Figure 12 for the placement of the reaction frame from through the Barge Moon Pool.

As the CPT is advanced the operator is provided a real time display of the records prepared for report presentation. Of importance warning signals are provided should deviation or excessive loading of the CPT occur upon encountering obstacles during the drive. Refer to Figure 13 for the real time records.

Following completion of the test the specified cone and friction resistance loads and the friction ratio is presented. Refer to Figure 14. A colour interpretation of the type of soil encountered, which include granular or cohesive layers, is provided. Refer to Figure 15.



Figure 11: Wheel drive unit



Figure 12: Placement through the Barge Moon Pool



Figure 13: Dual Wheel drive system

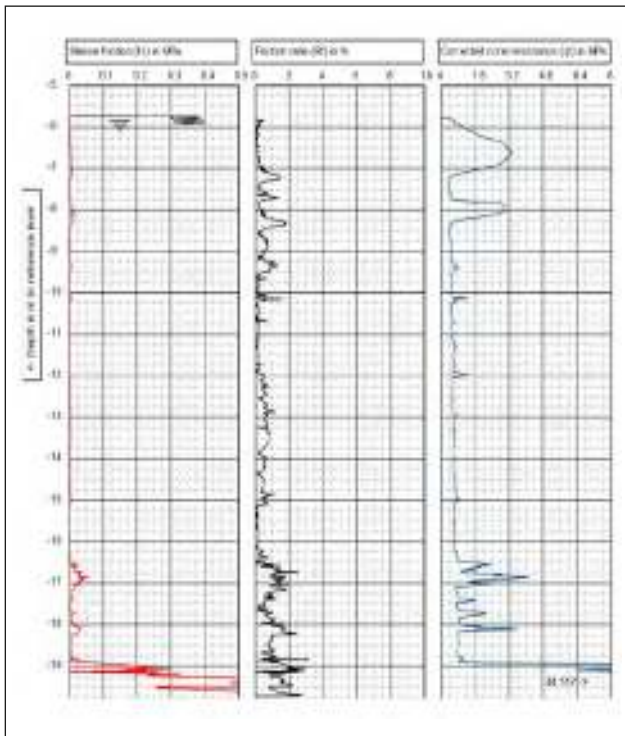


Figure 14: Cone & friction resistance and friction ratio

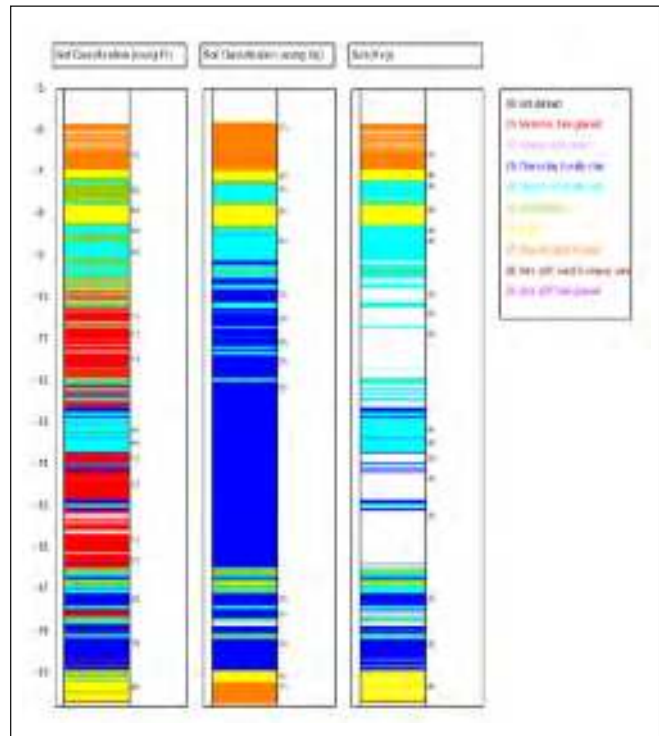


Figure 15: Soil Classification interpretation

CONCLUSION

The CPT has been used as a standard Ground Investigation technique since the mid-1930s. Since its original use from the mid-1970s in the HKSAR it has become widespread with the use of more recent Seabed CPT Systems allowing rapid mobilization to assess offshore geotechnical data for design input for ground improvement. In particular the use of Seabed CPT Systems, such as the GEOMIL 100 systems, has shown advancement in rapid placement, real time recording and clear report presentation and interpretation to accelerate the reclamation design and construction decision process.

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Any opinions expressed in this paper are those solely of the authors and not of any other party.

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A New Approach to Modelling the Ground

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ABSTRACT

Geological modelling is the first step to evaluate ground condition in the mining and geotechnical engineering industry which directs project planning, design, construction and cost estimation. Conventional explicit geological modelling involves time-consuming manual operations and raises difficulties to manipulate complex ground conditions. The modelling is essentially produced from subjective interpretations and is highly non-reproducible. Its workflow normally involves numbers of manual input and operations among different software programs. Such workflow cannot be dynamically updated upon amendment of design or supplement of geological information. With the advance of interpolation technology, 3D implicit geological modelling software programs vastly facilitates the workflow of geological modelling by modelling geological surfaces automatically and dynamically using mathematical algorithms. 3D implicit geological modelling software programs also allow modelling multi-source data, storing user-induced assumptions separately from raw data, and exporting conditional models on one platform. They provide advantages over traditional explicit modelling by minimising manual digitisation through a dynamic and automatic system. They also increase reproducibility through modelling from mathematical algorithms and separating interpretations from data. This paper presents a generalised workflow of project-specific geological modelling, then details a workflow of dynamic implicit modelling using commercially available software programs.

1 INTRODUCTION

Constructing geological models is one of the crucial steps in site investigation for assessing the site-specific geological conditions for engineering and mining, and investigating site characteristics which may affect the design, construction and performance of the works. An authentic geological model plays an imperative role on ground analyses and predictions which directs the subsequent planning, designs, practices and cost estimation (Fletcher et al. 2000). However, the traditional explicit modelling is time-consuming due to intense manual operations, involvement of individual interpretations, and heavily repetitive workflows upon phased data availability and amendments of project layout. Conventional modelling practices utilise various software programs which are not designated for geological modelling, bringing about risks of data inconsistency and errors during manual conversions and transfers of data.

With the increasing availability of data and complexity of project layouts, there is an apparent need of transforming the modelling methodology to accelerate the modelling processes by minimising manual, repetitive tasks using 3D implicit geological modelling software. This paper presents generic processes in 3D geological modelling, then proposes an improved workflow of project-specific and dynamic geological modelling which is tailor-made for a project.

2 GEOLOGICAL MODELLING APPROACH

2.1 Traditional Explicit Modelling Approach

The traditional 2D or 3D wireframing modelling generally refers to a Computer-aided design (CAD) based drawing process that relies intensely on manual digitisations. In general, stratum boundaries are constructed by digitising geologists' delineations of each stratum in serial sections, then the 3D strata are generated by joining the serial lines and undergoing triangulation. This methodology involves heavy manual digitisation and information transfer, which is firstly defined as "explicit" by Cowen et al. (2003).

Various modellers (Cowen et al. 2003; McLennan and Deutsch 2006; Turner 2006; Cowen et al. 2007; Birch 2014; Deacon 2017; Lane 2018) commented that the traditional CAD-based modelling is hard to model the complex geological features. Features like hanging structures cannot be well-generated using the solid modelling in CAD (Natali et al. 2013). Major limitations of explicit modelling include the following:

1. The workflow of geological modelling is complex involving various software programs. Multiple data transfers and conversions are prone to risk of data error.
2. Manual digitisation and repetitive workflow are time-consuming with potential human error.
3. 3D Modelling from 2D sections is hard to manipulate complex geology like faults, intrusions, marble with cavities and abrupt changes in stratum geometry, provoking high uncertainty between sections.
4. It is rare to hypothesise alternative geological models due to the time-consuming modelling workflow in the traditional approach. Areas of uncertainty in the model cannot be located and comprehended.
5. Explicit by different geologists could be highly variable and hardly be reproduced, leading to unknown risks from bias.
6. Models containing explicit triangulations cannot be automatically updated when there is amendment of the project layout or supplement of geotechnical data.

2.2 Implicit Modelling Approach

Initially introduced by Cowen et al. (2003), 'implicit' modelling is a process where a model is interpolated by a continuous volumetric function, which has been further defined and investigated by various authors over the years (Cowen et al. 2011; McLennan and Deutsch 2006; Cowen et al. 2007; Cáceres et al. 2011; Birch 2014; Hooykaas 2014; Basson et al 2016; Deacon 2017; Lane 2018). It models data efficiently using the fast radial-basis interpolation function (RBF). Such function allows data estimation from scattered 3D data of varying density, which is considered as one of the most suitable functions to interpolate drillhole data and model smooth geological surfaces (Franke 1982; Cowen et al. 2002).

McLennan and Deutsch (2006) illustrates the difference between explicit and implicit surfaces by Figure 1. It demonstrates a manually delineated stratum boundary by geologist in explicit model which varies with different individuals. On the other hand, the stratum boundary between two drillhole data in implicit model is interpolated by a mathematical function and only varies with different and known interpolation settings.

Implicit modelling vastly speeds up the workflow with the use of mathematical algorithms and advantages over the traditional modelling by the following:

1. By eliminating manual digitisation, it saves much time in modelling an immense amount of data.
2. It interpolates data by a volumetric function, producing models which are more objective and reproductive by other geologists.
3. As geological units are modelled as 3D blocks, it allows more accurate downstream manipulation of volumes and cost estimations for projects like excavations and resource estimations.

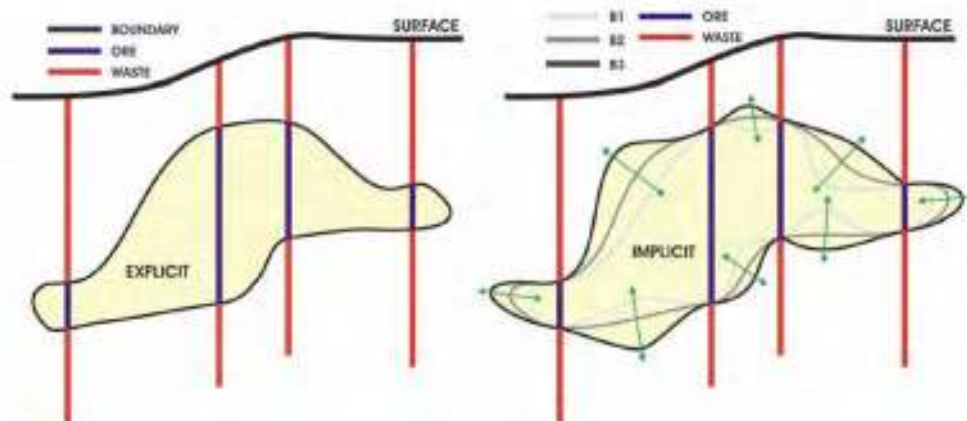


Figure 1: A schematic example of an explicit surface model (left) drawn by a geologist's assumptions and implicit surface model (right) by a mathematical function with known interpolation settings (McLennan & Deutsch 2006)

4. It separates user-induced assumptions and interpretations from raw data by introducing polylines, polygons or control drillholes for interpolations, giving more auditable models.
5. Models derived from fast mathematical function can be automatically and dynamically updated upon amendment of the project layout or supplement of geotechnical data.

Geological modelling involves multi-source, geological assumptions and integration of strata based on their contact relationship. The model will also undergo stages of refinement upon availability of additional drillhole data and geological information (GEO 2007). Therefore, it is indispensable to develop a simple and dynamic modelling method for fulfilling the need of the mining, petroleum and construction market. Such ground model development is supported by the commercial 3D implicit geological modelling software.

3 WORKFLOW OF A PROJECT-SPECIFIC GEOLOGICAL MODELLING

3.1 Generic Workflow of 3D Geological Modelling

Different projects have focuses of different geological interests and risks. Hence, the ground model should be tailor-made to exhibit different emphasis which suits the project needs (GEO 2007). For instance, site formation works may concern the locality and geometry of adverse geological structures and compressible soil, foundation design would matter potential underground obstructions during piling works, whilst mining projects are interested in ore deposits and fault systems for grade estimations. Although these projects desire different geological interests, the major steps in the modelling workflow are largely shared.

Various geological modelling workflows have been proposed (Cowen et al. 2002; Turner 2006; Shao et al. 2011; Pan et al. 2012; Natali et al. 2013; Song et al. 2018). A generic workflow of a project-specific 3D geological modelling is developed in Figure 2 modified from Shao et al. (2011) and Pan et al. (2012). It generally comprises five steps: (I) Data Preparation, (II) Data Processing, (III) Data Modelling, (IV) Model Refinement and (V) Model Application after the appreciation of project purpose and proposed works.

3.2 Workflow of Dynamic 3D Implicit Geological Modelling

A potent geological model requires multi-source data and modeller's interpretations of the relationship among geological units. As additional geological information was obtained and/or the proposed project layout was modified throughout stages of a project, the geological model would be updated with gradual increase in accuracy and precision of cost estimations (Wu 2005). This makes the traditional workflow of geological modelling highly repetitive and time-consuming.

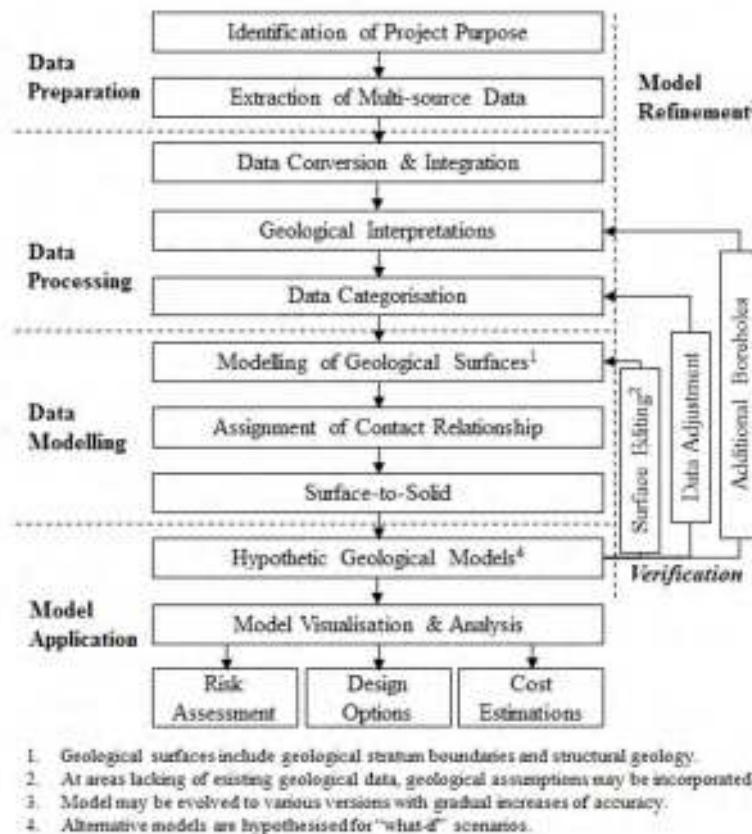


Figure 2: Generic Workflow of Project-specific Geological Modelling
 [modified from Shao et al. (2011) and Pan et al. (2012)]

A semi-automatic and dynamic geological modelling workflow is proposed (Annex 1) which can substantially accelerate the modelling process through semi-automatic and dynamic operations. This is maintained by two major systems: (I) Spatial Data Infrastructure (SDI) and (II) 3D Implicit Modelling System. The former system is a geographic information system (GIS)-based database which enables to store, query and extract any forms of geotechnical data spatially, whilst the latter system involves implicit modelling techniques which can centralise, model, analyse and export data in a dynamic manner.

3.2.1 Data Preparation

Geological modelling requires a review and interpretation of geological data from a variety of sources for greater completeness (Wu 2005; Pan et al. 2012; GEO 2007). Geotechnical data may be categorised into three types: Direct, Indirect and Auxiliary – the latter one demands more geological interpretation and data conversion than the former one. Modelling software with high compatibility is therefore an essential element for minimising manual operations in data conversion.

Direct data refers to data from direct observation, testing or surveying of samples from drillholes, pits or field mapping. Indirect data may include geological and geotechnical maps and graphs produced with assumptions by geologists. Auxiliary data could be sourced from any data that assists inference, interpolation and extrapolation of available geological information. It may include aerial photographs, historical maps, satellite images, topographic maps, DEM of topography, etc.

Since different projects have different focus of interests, the modeller should first appreciate the project objectives and defines area of study. Once a study area is digitised as a shapefile or CAD file, relevant drillholes can be selected from point shapefiles using ArcGIS. To optimise the process of data preparation, it is ideal to manage all the existing geotechnical data listed in Annex 1 in a SDI in which data of any format is georeferenced in GIS. Then, all the relevant direct, indirect or auxiliary data can be automatically queried and extracted from the SDI using the digitised study area.

3.2.2 Data Processing

Multi-source data needs format conversion to be integrated into a modelling program which can potentially be automated through scripting. Recent implicit geological modelling software programs are compatible for wide variety of formats like text file, AGS file, gINT project file, GIS vector file, 2D/ 3D CAD file, grid file, mesh file, georeferenced images, and other formats.

Geotechnical data input for modelling have to be reviewed before using. Some of the major issues arising from multi-source data integration may include: (i) choice of spatial reference, (ii) numerical data accuracy, (iii) attribute data accuracy, (iv) numerical precision, (v) data integrity and (vi) consistency of geological interpretation in logging, surveying and testing results. Item (i) can be automatically transformed with the aid of GIS software programs or other tools. Built-in data checking system in implicit geological modelling software can also be applied to validate item (ii). The most time-consuming parts lie on validating items (iii), (v) and (vi) which demand more manual inputs and geological judgements.

Before modelling, the raw geotechnical data ought to be simplified by categorisation. Categorisation groups the raw data for forming a strategic model to highlight key information necessary for further assessments which suits project needs. The emphasis and presentation of geological model therefore varies with project type using the same set of raw data. For instance, at the same site, data for seismic assessment would be grouped by grain sizes and soil stiffness; data for foundation project would be grouped to locate engineering soil, rock cores, weak zones (i.e. cavities and faults) and engineering rock; whilst data for a general overview could be grouped into superficial deposits, in-situ soil and rock. Figure 4 illustrates an example of how data at a construction site could be categorised for a foundation project after modeller's data interpretation, validation and categorisation. The data is then ready for modelling ("Modelling Data").

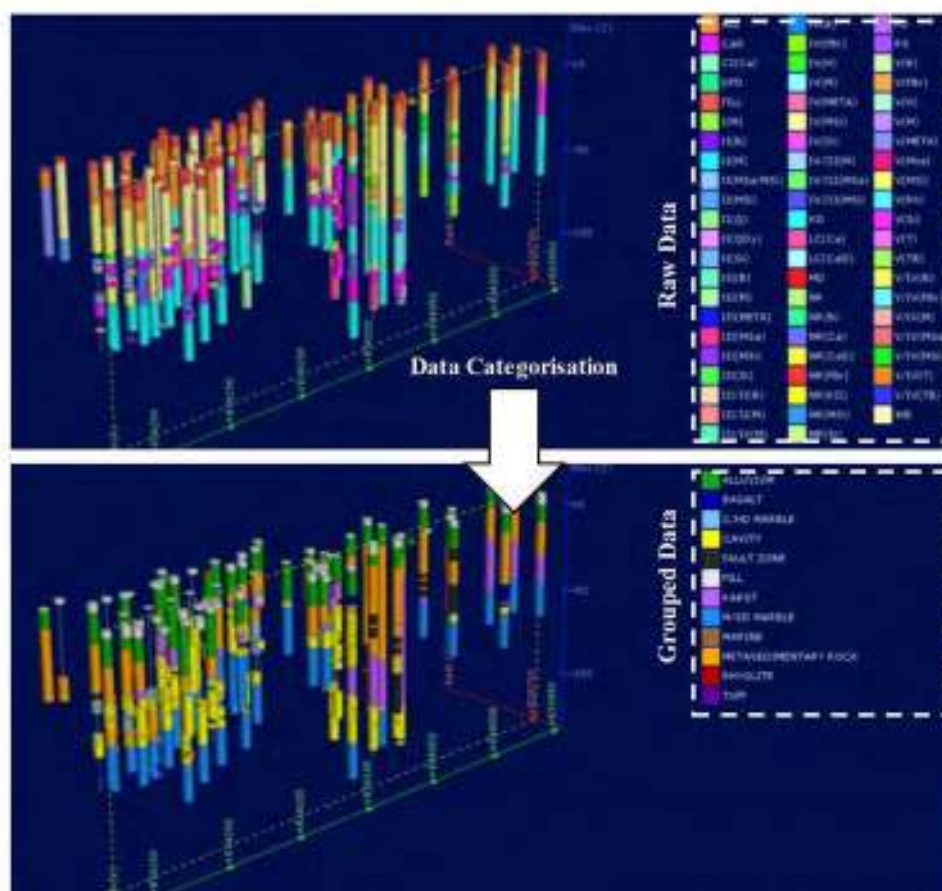


Figure 4: Geological data categorisation in a 3D implicit geological modelling software

3.2.3 Implicit Modelling

3.2.3.1 General Procedure

There are three intrinsic procedures in geological modelling: (i) Surface Modelling, (ii) Assignment of Contact Relationship, and (iii) Surface to Solid. The first procedure, Surface Modelling, includes three parts: (i) Topography Modelling, (ii) Geological Strata Modelling, and (iii) Geological Structure Modelling (optional). Data can be centralised, modelled, analysed and exported in a single platform using 3D implicit geological modelling software. Such software packages enable an efficient and dynamic system using mathematic algorithms to create model from ample data and instantly update it upon data amendment. Geological concepts can also be incorporated using various functions designated for geological modelling, like grouping lithologies, setting lithological chronology, and creating geological features like intrusions, veins and faults.

3.2.3.2 Topography Modelling

Topography is an essential information to shape the geometry of the geological model. This can be imported into the software as point data, contours or digital elevation model (DEM) in form of text file, GIS vector files, or grid file, then modelled by triangulation. An orthophoto can be overlaid on the DEM for visualisation.

3.2.3.3 Geological Strata Modelling

In 3D implicit geological modelling software, boundary surfaces of each geological stratum (i.e. each category of the model data) can be created by interpolating its top or base level. Faults and stratum boundaries of different formation natures can be generated by the software to suit the site's geological settings. The modeller should comprehend the characteristics each surface modelling functions in the implicit geological modelling software and select one that best exhibits the geological features at the site based on their formation nature.

3.2.3.4 Geological Structure Modelling

At sites with complex structural geology, fault is a key consideration in projects like cavern, tunnel, foundation and ore and resource estimation. A fault model can be built by modelling fault zones as a linear surface, or by modelling brittle faults using the fault system modelling function in the software. Fault information used for the modelling can be in form of drillhole data, GIS vector data, polyline or surface. Some software packages also support direct import of field measurements of structural data. Fault information can also be delineated as polyline by tracing the fault over a georeferenced geological map in the software.

3.2.3.5 Building the Solids

Since each geological and fault surface is interpolated independently, they may interpose with each other. To integrate the modelled surfaces, their contact relationship and formation nature should be assigned based on geologist's judgements from geological maps, related reports, and field observations. Such integration is traditionally handled by manually erasing strata conflicts spotted by eye in CAD, and repeats again during re-modelling. However, the integration can be done automatically in implicit geological modelling software.

Such software programs enable to incorporate geologist's interpretations by assigning the age relationship of the modelled strata and faults. Batch volumes of the selected strata can be generated simultaneously to form a preliminary 3D geological model. In this process, geological solids are generated based on user-defined age relationship and formation nature of each geological unit. This eliminates the repetitive process of manually integrating the surfaces, which remarkably expedites the modelling workflow during model refinement.

3.2.4 Model Refinement

A major issue encountered by modellers in implicit modelling is having limited control on the surface modelling (Natali et al. 2013). The parameter settings in the implicit algorithm may not produce a surface that coincides with modellers' expectations at areas with limited raw data. Analysis by Deacon (2017) has also acknowledged a direct impact of data density on modelling discrepancies between explicit and implicit modelling. Therefore, it is imperative to verify and refine the implicit model with geological judgements, particularly at areas with sparse or no data. When project-specific drillhole is unavailable at the early stage of project, an optimal modelling approach is to incorporate explicit modelling into the implicit model through supplementing user-induced assumptions for enhancing model's realism (McInerney et al. 2007; Natali et al. 2013). By introducing assumptions into the implicit model, alternative models may be hypothesised to illustrate the uncertainty inherent in the model.

In implicit modelling software, modelled surfaces may be refined by introducing assumptions as user-defined geometry (polyline; polygon) or control drillholes. Alternatively, the modeller may refine by changing the settings in interpolation and data snapping. It may happen when, for example, unreasonably thick colluvium is modelled at spurs where no drillhole is available. The layer may be re-modelled by hand-drawing a polyline or inputting control drillholes such that the deposit conforms to the topography. In another case, when fault information is insufficient from drillholes, fault information may be incorporated into the model by delineating a hypothetical fault plane on a georeferenced fault map. Then, alternative models and cost estimations can be proposed by hypothesising possible dipping directions. Cowan et al. (2003) also demonstrated a user-defined grade continuity in which the grade orientation is constrained by two hypothetical planes of opposite dipping directions, hence proposing two alternative models and grade estimations.

Assimilating user's assumptions by these means allows to record individual's interpretations separately from raw data. This makes the model and modeller's interpretations profoundly reproducible and auditable (McInerney et al. 2007). In implicit modelling, geological volumes can also be re-interpolated instantly once user-defined assumptions are added, freeing time for hypothesising alternative models for risk management.

When project-specific drillholes are available and imported into the software, implicit modelling can dynamically re-interpolate and update the model. The user may then immediately verify his or her assumptions, modify the user-induced data, and produce a more promising model. Data inconsistency can also be easily spotted in the 3D and dynamic view in the software which greatly facilitates the verification process. When there is, data may require adjustment by ignoring illogical data or editing data categorisation. The model is then instantly updated by the implicit modelling engine, minimising manual and repetitive tasks.

Early-recognition and communication of ground uncertainty and unfavourable conditions followed by regular model refinement are considered good modelling practice for a cost-effective delivery (GEO 2007). Presenting uncertainty in alternative models also avails strategic ground investigation planning and reduces risk and cost from 'unforeseen ground conditions' during construction.

3.2.5 Model Application

Our recent trial shows the geological model generated by 3D implicit geological modelling software (Figure 5) exhibits more confident results than that generated by traditional modelling practice in view of minimal manual inputs, elimination of repetitive tasks, reduced risk of human error, and dynamic model update upon additional data. This tremendously reduces time and cost in modelling. After constructing a model, the capability of extracting vital information is a key to effective communication among different analysts and audience. High interoperability of implicit modelling software is essential to export a wide variety of information from the model for visualisation, various risk assessments, design options and cost estimations.

Commercial 3D geological modelling software packages are generally compatible with gINT, ArcGIS, and CAD. To illustrate the geology at proposed works, the model can be exported as a 3D image (Figure 6a) or serial cross sections in PDF or CAD files (Figure 6b). The modelled solid, surfaces and drillholes can also be exported as 3D CAD files (*.dxf, *.dwg, *.obj, *.lfc and *.dgn) (Figure 6c) to integrate with

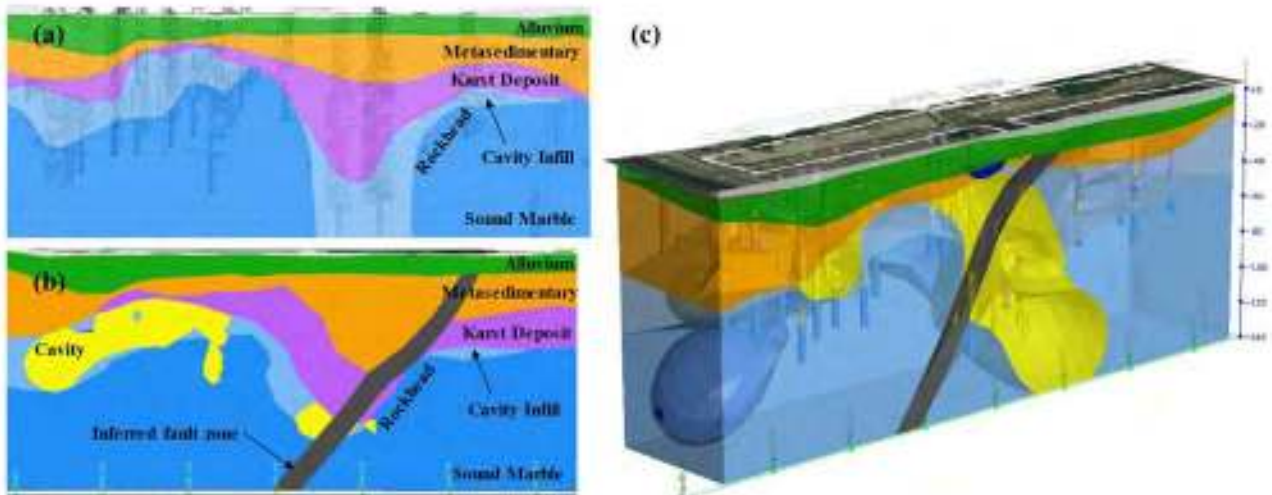


Figure 5: Geological model generated by traditional practice (a) and 3D implicit modelling software (b) & (c)

information of other disciplines and conduct further assessments and submissions using building information modelling (BIM). This also allows room for automating geological-dependent designs in BIM, like pile design may be automatically updated upon amendments in geological model based on a prescribed engineering logic.

Statistical analysis of modelled data in the software can simultaneously export the geological parameters relevant to the project. Volumetric estimation of soil/ rock and cut/ fill is indispensable in projects like tunnelling, quarrying, and site formation for cost estimation and cost-effective design. 3D implicit geological modelling software is an intrinsic solution for accurate and rapid volumetric analysis. By combining the geological model with a 3D CAD file of the proposed works (Figure 6a), volumes and other parameters of the modelled geological units within proposed works can then be manipulated for other engineering modelling.

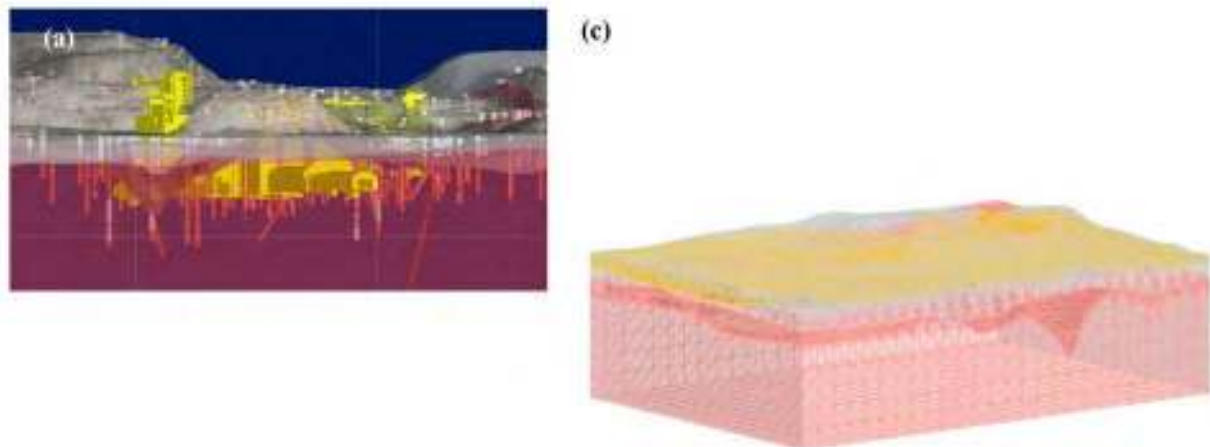


Figure 6: Data output from 3D implicit geological modelling software. (a) An image illustrating the interface between the model and project design in yellow which is imported as a 3D CAD file. (b) Batch export of serial sections at proposed layout. (c) A 3D DWG file of geological model from implicit modelling software imported into a BIM environment.

4 CONCLUSIONS

With the expanding demand and complexity of projects, technical advancements in 3D geological modelling become a current interest among the mining, petroleum and construction engineering markets. There is a gradual transformation of modelling workflow from a CAD-based system to a more automatic and dynamic system which drastically minimises manual operations since the release

of implicit geological modelling software programs. As geological model continuously evolves with supplement of data or amendment of project layout, implicit modelling can immensely reduce time and cost of project business.

A project-specific 3D geological modelling workflow is proposed in this paper which consists of (I) Data Preparation, (II) Data Processing, (III) Data Modelling, (IV) Model Refinement, and (V) Model Application. A semi-automatic and dynamic system is maintained in this workflow by two systems: (I) Spatial Data Infrastructure (SDI) and (II) Implicit Modelling Software. This requires an implicit geological software which supports multi-source data, data centralisation, data modelling and data exporting on a single platform.

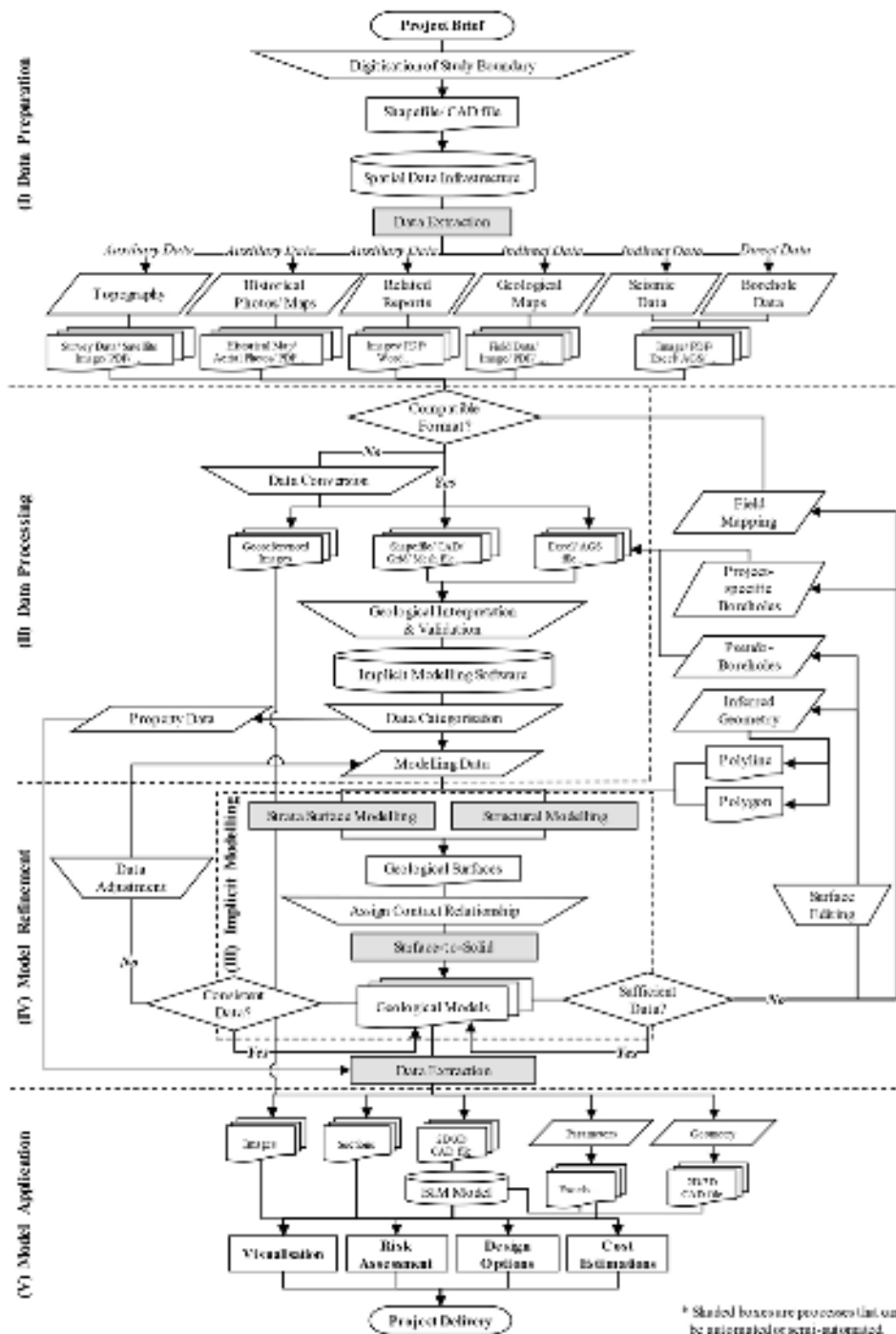
A shortcoming of implicit modelling is a mismatch between the implicit model and geologist's expectation at areas with sparse or no data. Hence, the author recommends an optimal modelling approach – to incorporate the modeller's interpretations into the implicit model by re-modelling the surfaces with user-defined control drillholes or geometry. This can separate individual's interpretations from the raw data, giving highly reproductive and auditable geological model.

A simplified and dynamic geological modelling workflow tremendously saves time and cost in project delivery. It frees time to hypothesise alternative models based on different geological assumptions to illustrate the uncertainties inherent in the model, proffering a low-risk and high-accuracy design in a cost-effective way.

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Annex 1: Workflow of a semi-automatic and dynamic 3D implicit geological modelling [modified from Cowan et al. (2002), Wu (2005), Turner et al. (2007), Shao et al. (2011), Pan et al. (2012), and Natali (2013)]

Construction 4.0: Risk Reduction and Productivity Improvement through the use of Expert Systems

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ABSTRACT

Construction is a rapidly changing environment which accumulates ever increasing quantities of data through the activities of a diverse collection of people and machines. The effective processing of these data into valuable timely information and the communication of this information across the project population remains a challenge and these difficulties ultimately pose significant risks to achieving successful project outcomes.

MissionOS has been developed over the last 14 years and provides a single integrated platform to connect specialist staff and systems throughout the complete lifespan of a construction project from planning through to post construction performance monitoring. The MissionOS is configurable by clients to address any form of data but its structure has been designed entirely around the infrastructure planning, design and construction process. The entirely web-based MissionOS benefits from various advanced programming technologies to enable users to create expert systems which promote group and machine learning, increase the efficiency and productivity of projects and reduce risk.

We will describe the application of MissionOS on several recent projects throughout Asia including Liantang Boundary Crossing, Kuala Lumpur metro and Singapore's Power tunnels highlighting particular applications where technical advancements, risk reduction and productivity improvements were achieved.

1 INTRODUCTION

1.1 Construction 4.0: Incentives to reduce risk and increase productivity

Construction 4.0 is the Engineering and Construction equivalent of Industry 4.0, a move towards greater digitalisation in the pursuit of greater productivity and quality with reduced time, cost and human risk. This covers a variety of new technologies including the use of robotics and drones, sensors and virtual reality but above all it is driven by enormous quantities of data. The keys to future success lie in the ability to improve the quality of this data and to use it in real time to guide decision making within the fast-changing construction environment. There are significant challenges.

Industry 4.0 focuses on automation of repetitive processes producing standardised products, but in construction each design is unique so building in the same level of efficiency requires flexibility and configurability. Once data is collected it can improve the efficiency of production through constant feedback and with consistent structured data there is the opportunity to build machine learning and expert systems to automate the process of analysis. In addition to improving active projects such structured data sets add value to the planning of future projects and raise the collective knowledge base of those in the industry such that risk in construction can reduce progressively.

To meet these ends the structures must be carefully designed to provide information that is fit for purpose and which also has longevity. This paper reviews the role of cloud-based data management

systems within the framework of Construction 4.0 and describes ways in which the MissionOS has been developed to provide the structured data needed and the facilities to handle expert systems.

2 THE MISSIONOS PLATFORM

MissionOS is a data management platform hosted on the cloud with built in structures which are ideal for the storage and analysis of construction data. It has been designed to fit the requirements of infrastructure project management throughout the projects lifetime and as a basis for business to maintain data across multiple projects and seek greater business efficiencies.



Figure 1: MissionOS Control – DTSS2 Singapore

The MissionOS tracks data through projects from planning to asset management through five main areas:

Table 1: MissionOS data areas

Stage	Data Area	Description
Investigation/Planning	Influences	Geology and geotechnics, environmental constraints, ecology and habitat, hazards, susceptibilities
Design	Predictions	Ground, structure and temporary works movement and stress strain predictions. Construction performance predictions (trough width parameter, cutter wear rate), ground impact predictions (water inflow, grout take, temporary and permanent support)
Construction	Production	Progress, Programme, Activities, Metadata eg. Inflow, Grouting, Geology, Support, Concrete deliveries, ring construction, concrete placement. Etc etc
Construction	Feedback	Survey, geotechnical and structural instrumentation, machine monitoring (TBM, Drill Jumbo), environmental monitoring, safety monitoring
Operation	Performance	Structural health monitoring, maintenance management

2.1 Platform design

The objective of the system is to provide a platform for all data and provide users with the flexibility to design analysis methods and reports as required with functions focussed on engineering analysis. The data can be divided into the following types.

Table 2: MissionOS data types

Data Type	Description
GIS Data	Layers of geographically positioned data unlinked to construction jobs (maps, boreholes, hazards, existing sensitive structures, slopes. Terrain models, INSAR, point cloud data, 3D CAD models, photos, commentary, reports
JOB Data	Programme, geometry, progress, time and activities, metadata linked to activities
SENSOR Data	Measurements of machines, sensors in the ground and on buildings and structures
EVENT Data	Responses, photos, decisions
ENVIRONMENT Data	Video, sensor data, weather, tide data

The platform is completely configurable, and users can design models for each of these data areas including relational data fields, constants and calculations. Loop calculations which iterate along a chain of sensors can be created to define compound and derived instruments such as inclinometers and distortion profiles. Non sensor data can also be loaded in the form of ground investigation data such as AGS or CPT data and also in the form of predictions using the results of numerical analysis. With its ability to perform complex calculations, MissionOS can also define exemplar models for geotechnical and structural behaviour which can then be ‘tied’ to real data points.

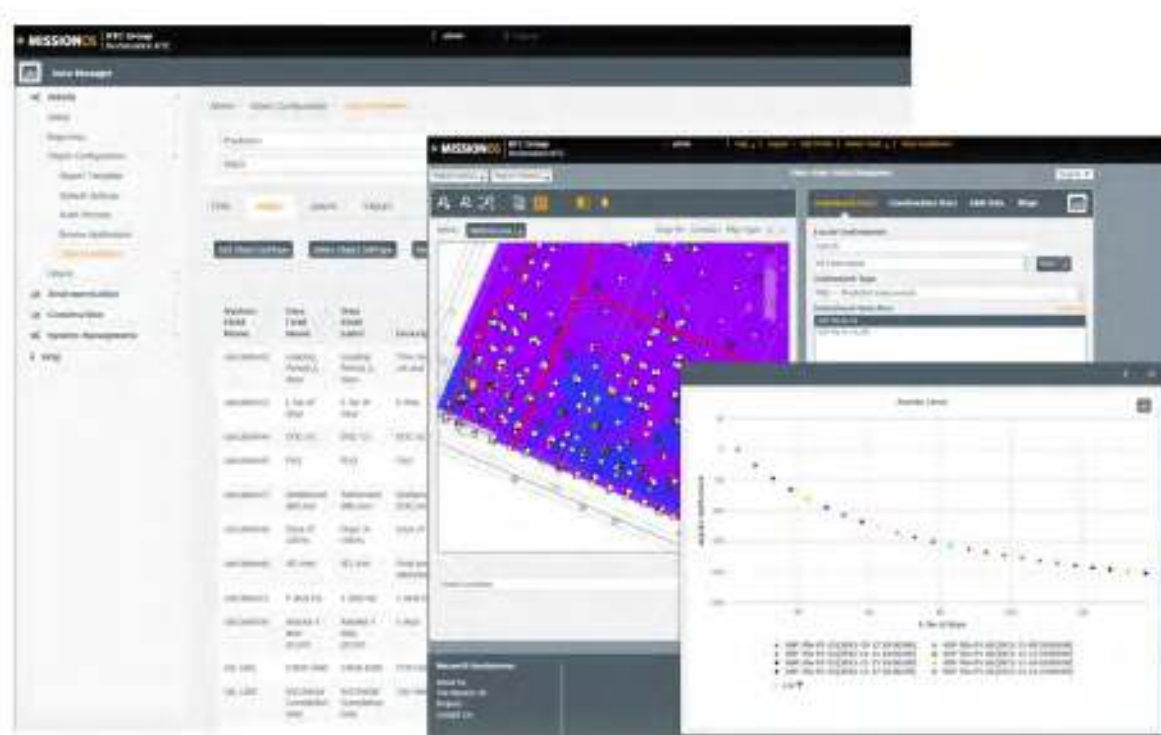


Figure 2: Asaoka exemplar model constructed in MissionOS for ground improvement

Job data is derived from the programme with jobs divided into jobs, subjobs and fragments.

Table 3: Job data structure in MissionOS

Job Structure Type	Description	Example
Job	If it is done as a single job then has a shift report. If split into several fronts ie fragments then will not have a shift report. Has a measurement of progress which may be linear or a sum/count of fragment progress	Piling (progress is the sum of the number of piles completed in zone A and zone B)
Fragment	A part of a job which is undertaken at the same time as another part and each have separate shift reports	Piling Zone A Piling Zone B
Subjob	An job which is too small for an individual shift report but which is tracked on the Job shift report as a gantt bar	Pile A1, A2 etc
Activity	The activity underway – this may be linked to a subjob progress	Eg Concreting
Metadata	Each activity can be assigned a number of defined metadata models	Cube tests, slump tests

3 MISSIONOS AS AN EXPERT SYSTEM

The structure detailed above lends itself well to application as an expert system. Based on available data, a prediction for job behaviour is made in advance. During the progress of the job new data is added and the prediction data set can be updated, and new predictions derived. Sensor data and metadata is also collected during construction and tracks the deviation of real behaviour from the model. MissionOS then helps the user to alter the models to give better forecasts of behaviour in the future. These models and the data behind them are then pooled and re-used for the planning of future jobs thereby reducing risk on individual projects and on the whole business.

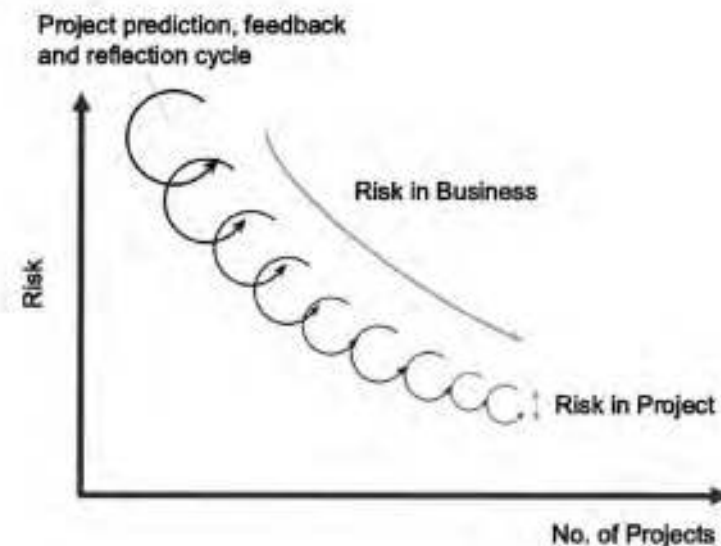


Figure 3: Progressive Risk reduction through MissionOS Implementation

4 MISSIONOS EXPERT SYSTEMS IN RISK MANAGEMENT

Risk management in MissionOS focuses on a number of key areas organisational risks, physical risks, and commercial risks. These are addressed in MissionOS by promoting:

- The better use of data in the planning process, pre-, syn- and post construction;
- Recognising and managing uncertainty using real time feedback systems;

- Improving forecasting and job management through automatic reflection;
- Increasing confidence through better information flow;
- Enabling collaboration without prejudice.

4.1 The better use of data in the planning process, pre-, syn- and post construction

4.1.1 Harbour Area Treatment Scheme

The Harbour Area Transfer Scheme (HATS2) was the follow-on project from HATS1 or Strategic Sewage Disposal Scheme completed in 2001. One of the difficulties on HATS1 was the limited amount data upon which to interpret ground conditions at tunnelling level particularly as the tunnels were deep, under marine areas and alignments were changed at a late stage in design. During the project Hong Kong's first Tunnel Data Management System was constructed to manage data and this was used during the settlement of claims and during the subsequent phase 1 arbitration won by Government.

The data was handed to the consultants engaged in the feasibility and detailed design of phase 2. Rather than putting support, drilling and grouting within the excavation lump sum this data enabled the Phase 2 to be tendered with provisional quantities significantly de-risking the project for the contractors. Ultimately this was highly successful and provisional quantities came in within 5% of expectation. (Endicott pers comm, 2015).



Figure 4: Example of Cycle Series Data for HATS1 used for HATS2 Engineering feasibility

4.1.1 Cycle Series Data to Define Probability Distribution Functions

Figure 4 shows a representation of cycle series data extracted from the HATS Phase 1 data. In cycle series, data is summarised to a common basis the 'cycle' which has significance for the construction process. For example, this may be per m, per day, per blast, per probe/grouting round etc etc. Once in this form then the data can be interrogated to produce probability distribution functions (Figure 5) and tested to identify correlations with performance metrics.

The cycle series data rules required significant engineering input to be meaningful. It is often not sensible to simply average data across the different length units since this destroys the subtle relationships between the parameters. For instance, the maximum convergence in a tunnel is governed by the lowest Q material in a section and thus maximum movement against an average Q value would not give good correlations.

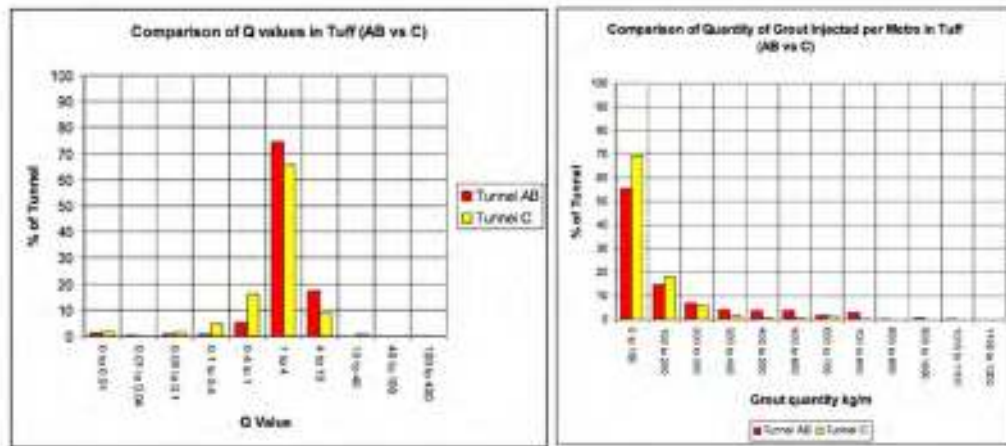


Figure 5: Probability Distribution Function for HATS1 tunnel data used in HATS2 planning

4.1.2 Rapid updating of geologic parameters on Liantang Heung Yuen Wai Control Point

On the Liantang Heung Yuen Wai tunnel MGS and Dragages undertook observational engineering for the initial drive from the North Portal. This was largely due to the uncertainty around the ground conditions and in particular the performance of the various weathering layers (Storry et al 2017).



Figure 6: The north portal of the Liantang Heung Yuen Wai Tunnel

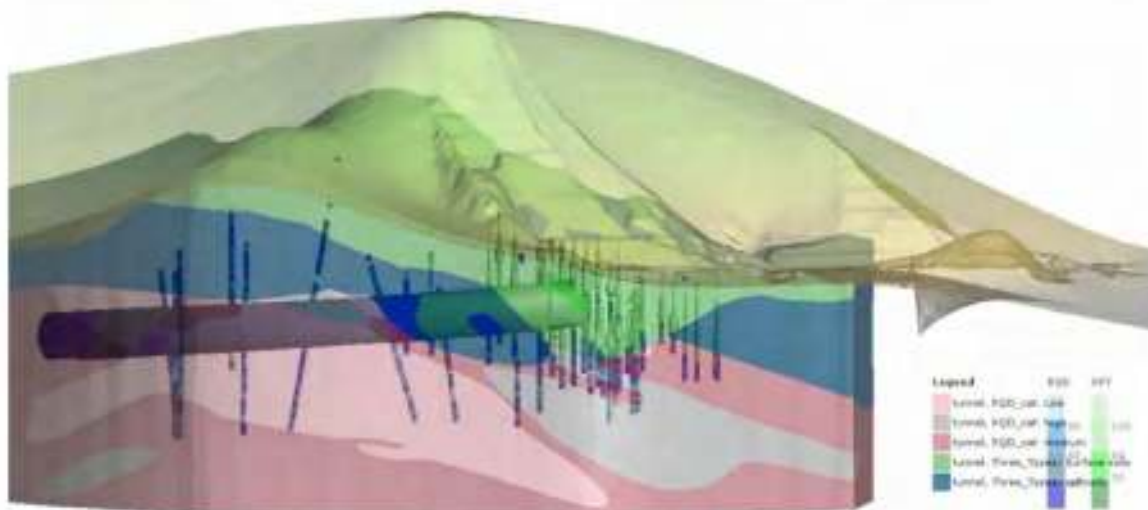


Figure 7: Data driven models of geomechanical units

As part of this process MGS engaged with Sequent (formerly ArantzGeo) to use their data driven modelling environment Leapfrog to evaluate bidirectional links to MissionOS to update geological profiles in real time.

Sections in leapfrog were automated and could then be updated into MissionOS at any time and evaluated against time series instrument and TBM data. This ensured that at all times the operators would have access to the latest models for the ground ahead.

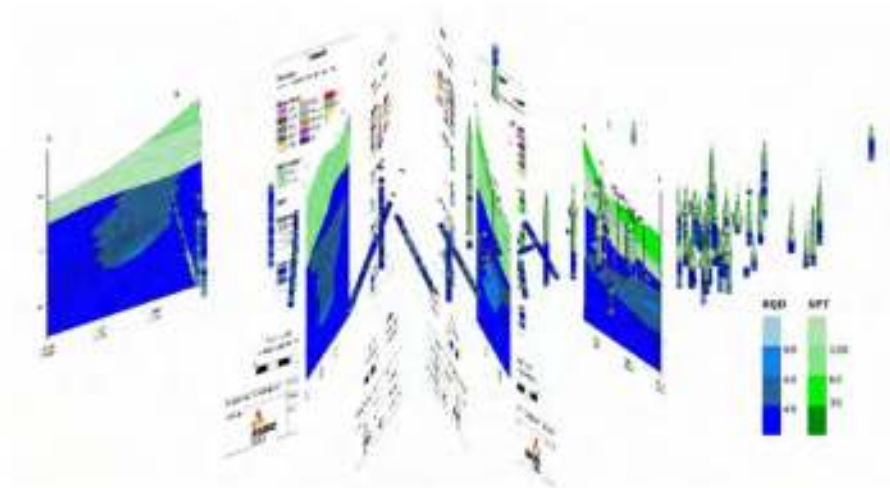


Figure 8: Automated cross and long section generation and input to MissionOS

4.2 Recognising and managing uncertainty through the use of real time feedback systems

On the recent Singapore Power Assets (SPPA) cable tunnels and the Klang Valley metro phases 1 and 2 (KVMRT) there has been a drive to provide live feedback on design assumptions to inform future prediction. The main areas for this are geology and its relation to TBM face pressure settings and intervention planning and pre-prediction of settlement in the immediate vicinity of cutterhead and tail-skin.

4.2.1 Geology and its relation to TBM face pressure settings and intervention planning

The capability in MissionOS to combine a large quantity of data into real time screens was used to evaluate the settlement response to the TBM excavation process including reaction to changes in excavation chamber pressure. On SPPA there was particular focus around the transitions between rock and soil where slurry loss could be expected and more difficult excavation may result in higher than normal volume loss.

This system was automated only to the degree that the dashboards were automatically prepared in real time and that targets could be set very simply by emailing the system when it appeared likely that a geological change was imminent. In this case the geological models were also fixed image backgrounds and were not updated with new data nor did they contain information on the confidence of the interpretation. With the use of real time data driven geological modelling to define the location of geological surfaces these changes can be predefined. The amount of the increase/decrease of face pressure required ahead of the machine was analysed by engineers and re-input however there is scope for this to be estimated based on machine learning if the historical final outcome data is analysed using the cycle statistics approach. The same applies to the estimate of target excavation volumes in different materials which is based on an assumption of bulking within the cutterhead.

Cutterhead interventions are also dependent on the location of the rock/soil interface with a need to avoid locations with heavy inflow or ravelling materials whilst optimising cutter wear.

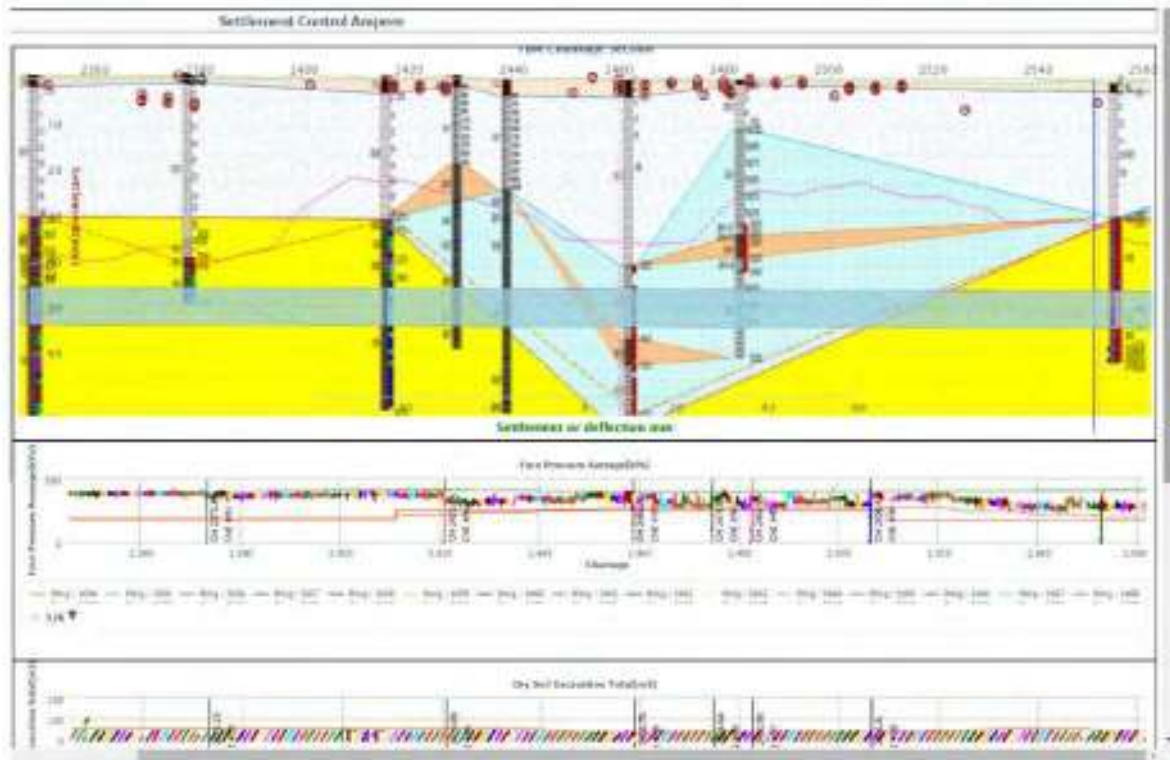


Figure 9: TBM control dashboards were used on Singapore Power cable tunnels to allow review of geology settlement and TBM parameters and the updating of machine targets.

4.2.2 Pre-prediction of settlement in the immediate vicinity of cutterhead and tailskin

Pre-prediction for settlement ahead of tunnel boring machines allows the actual settlement to be evaluated against the measured monitoring and if feedback is quick enough then mitigation measures such as changes of face pressure, soil conditioning and increase in tail skin grouting can be adopted. Figure 10 shows automated calculation of target gaussian curves using trough width parameters determined from investigation and design. Two type of point are created: A dynamic point which travels through the data and has three sections calculated; one diameter ahead of the face, at the face and two diameters behind the face. A set of permanent points with a single line showing the final settlement.

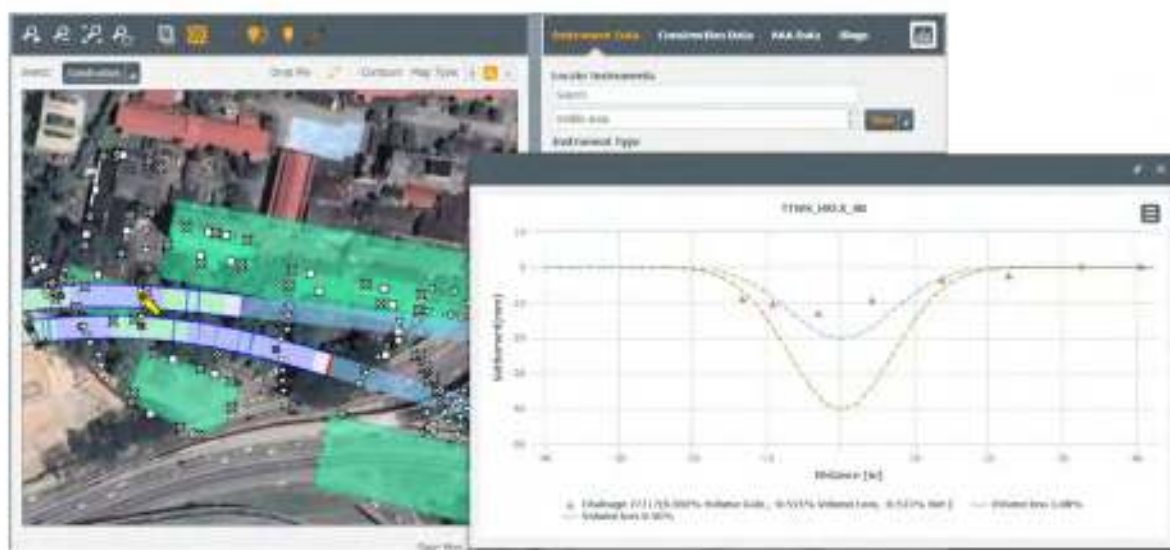


Figure 10: Dynamic evaluation of settlement against prediction

The actual ground loss is evaluated for these permanent sections corresponding to 1, 2 and combined tunnels. With this available it is possible then to reset the assumed trough width parameter for similar ground conditions ahead.

For best prediction of settlement, a longitudinal cross section is required which is linked to a real time function called proximity(jobname). This can be created within MissionOS as a pseudo-instrument using a chain of calculations and an independent variable covering proximity.



Figure 11: Proximity based dynamic prediction of settlement.

This model is then applied to each settlement point to be used for monitoring and site-specific parameters are applied to the forecast. These can readily be modified as more knowledge of particular ground conditions and responses are collected and are a future target for automated self-learning algorithms to be built in to the system.

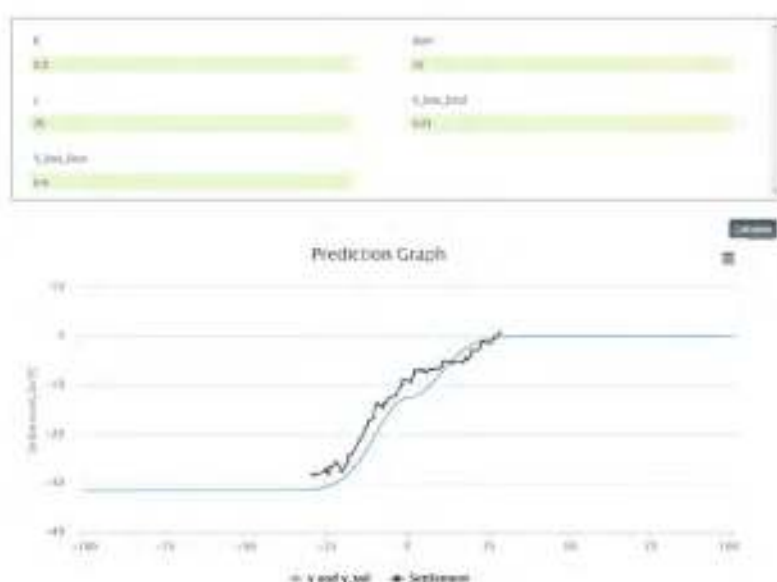


Figure 12: Site specific constants are applied to generate forecasts at each monitoring point.

5 CONCLUSIONS

There are many opportunities to apply expert systems into construction to manage daily decision making. With data collected in structured ways and readily processed into cycle series data we can now reflect and adjust in a highly automated way to arrive at optimum outcomes.

This is not without its difficulties. Figure 13 is a plot of settlement vs chainage where instrument dates have been converted to equivalent tunnel face chainage. For each plot the movement to the left of the triangle is movement prior to the arrival of the machine at each point and to the right is after the passage of the face. This demonstrates the difference between greenfield behavior and the realities of complex and dynamic tunnel/soil and soil/structure interactions. In this case the changing geometries of tunnel excavation beneath a rigid concrete structure has lead to unexpected but highly predictable outcomes. The classic double kinked settlement over first face then tail-skin is evident but as the tunnel is excavated around the curve the outer prisms actually heave before the tunnel approaches due to the rigid behavior of the structure.

As discussed in section 4.1 of this paper the condensing of information into usable forms can only be done with a knowledge of construction in all its facets; geologists are required to work with hydrologists and grouting engineers to determine which parameters are likely to correlate and how to best summarise data to achieve that correlation.

Such expert systems cannot function across a business unless a common data platform is put in place with common standards for how most of the data is collected. This should however be flexible enough to allow the standard to evolve over time but rigid enough to allow the fundamental cycle series data to be calculated. The platform also needs the tools to allow the end user to investigate the summarized data and to understand from where it is derived.

MissionOS is such a platform and has evolved over the last 14 years to look after all project data on several construction projects where clients and contractors have driven the requirements.

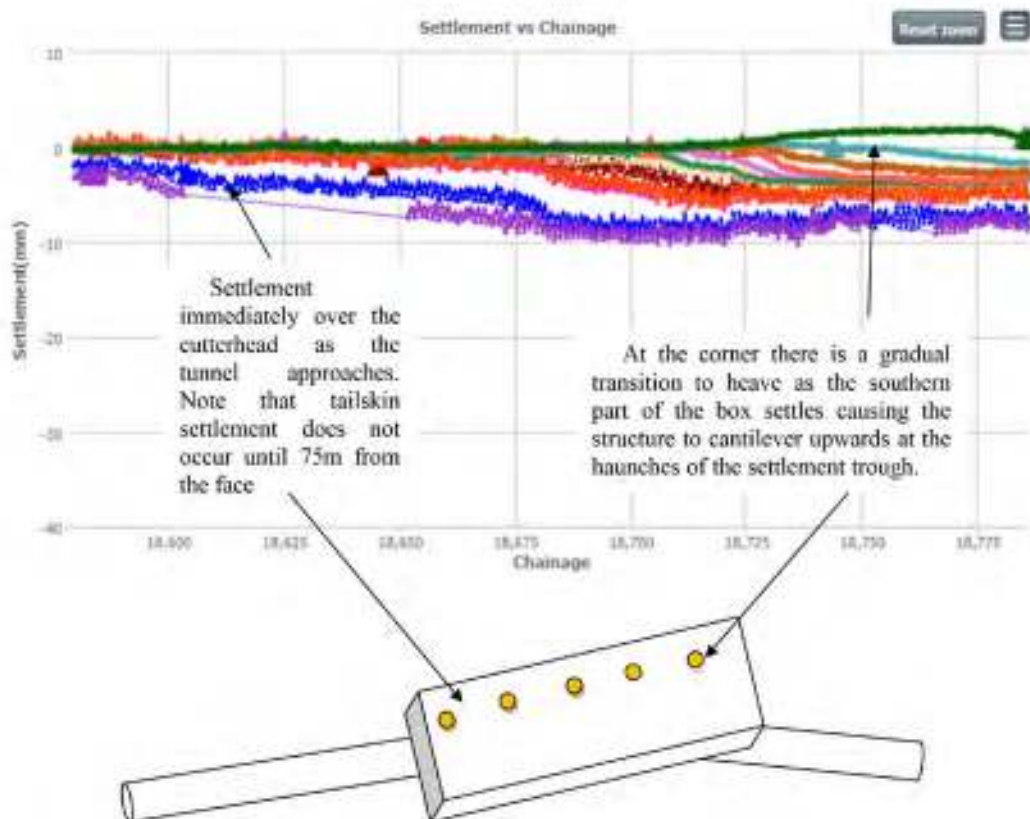


Figure 13: Tunnelling around a curve beneath a buried rigid tunnel box demonstrates the complexity of soil structure interaction in three dimensions.

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Full scale testing of rebar and SFRC segmental lining using independent and wireless instrumentation systems

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ABSTRACT

Development and acceptance of steel fibre reinforced concrete (SFRC) segmental tunnel lining requires an accurate understanding of the performance of the SFRC segments under real load conditions. An innovative instrumentation scheme has been implemented during the construction of the Liantang Lung Shan highway tunnels in Hong Kong to evaluate the distribution of the loads in the segments during erection and tunnelling and for comparison with the design assumptions obtained by numerical simulations. The monitoring system used embedded micro-electronic technology and was designed to be independent and wireless allowing data to be collected throughout the full lining erection process. The instrumentation program included 10 segments, both conventionally and steel-fibre reinforced. This paper will present the development of the instrumentation, the installation methods and initial data obtained.

1 INTRODUCTION

1.1 Main objective

An R&D project was implemented by Dragages Hong Kong and Bouygues Travaux Publics at the Liantang Lung Shan highway tunnels project in Hong Kong to study the structural behaviour of the steel fibre reinforced concrete segmental lining and to evaluate the distribution of loads in the segments in order to confirm design assumptions used in numerical simulations.

A comparison between numerical and experimental results will be carried out to assess the effectiveness of the adopted numerical model in predicting the behaviour of segmental lining under a thrust load simulating the effect of the TBM machine during excavation. Monitoring data enabling a comparison to be made between conventionally reinforced concrete and steel fibre reinforced concrete (SFRC) segments was also obtained for similar analysis.

The monitoring instrumentation was developed to record strain developed in the segment due to the segment handling, TBM thrust and the annular void grouting/ground loads.

Segment reinforcement ratio is generally defined during design by the grout/ground/TBM thrust loads hence, the strain gauges were positioned where it was predicted that the maximum strain would be developed due to these loads.

1.2 Specific development requirements

Prior to the development of the monitoring system several important criteria were defined which needed to be considered by the system developer; one of the most important development criteria considered was to be able to obtain data as soon as the TBM thrust pads are in contact with the segment and before the vacuum erector is removed. Literature studies indicated that previous segment monitoring schemes have not been able to capture this window of data. Due to access restriction during segment erection, the difficulty to plug cables to segments when the erector pad is still on the segment and also the inconvenience of having cables running in the tunnel that can be damaged by the traffic (TBM, MSV, cable bracket installation...), a wireless monitoring seemed the most appropriate technology to pursue.

Other factors to be taken into consideration during the development were the positioning of the instrument components, so as not to adversely affect the data obtained (i.e. affecting stress distribution within the concrete or introducing artificial lines of weakness within the segment), together with the frequency of monitoring and the duration of monitoring.

2 INSTRUMENTATION

2.1 Instrumentation development

Instrumentation using conventional vibrating wire strain gauges, multiplexer, datalogger and battery would be impossible to embed in the concrete due to the size of the elements. The power consumption of vibrating wire strain gauges would require big batteries which are also not convenient to embed within the segments. Collecting data through a multiplexer would also prevent synchronous data reading at the frequency required.



Figure 1: Foil strain gauges and strain gauge module (SGM), Figure 2: Data logging module (DLM)

Foil strain gauges (Figure 1) were considered as they are resistive and can be recorded at a higher frequency than vibrating wire gauges. Such gauges are commonly used for concrete testing and to monitor the performance of structures. However, these gauges are susceptible to variable signal loss in cabling and this is why measuring vibrating wire instruments in the frequency domain is often preferred. A new method which samples a resistor / capacitor time constant rather than simple voltage allows conversion of analogue sensor direct to microstrain without being affected by line losses. This new electronic arrangement requires much less power and therefore a smaller logging unit and power supply. As a result, the system works on regular batteries (8.5Ah battery C cell size, Figure 2).

An application was developed to communicate with the datalogger via Bluetooth and to download data. The application also enabled the data sampling rates to be configured on site and to manually read temperature, battery condition and strain gauge conditions.

The key points of the custom development were as follows:

1. The design used a very low-power strain gauge coupled with an RS485 transceiver.
2. The sensor nodes comprised 3 x ¼ bridge strain gauges.
3. The strain gauges were temperature compensated.
4. The strain gauges were wired directly to a nearby module containing signal conditioning circuitry and microprocessor.
5. The wiring configuration was designed such that there would be independent channels providing redundant paths to all strain gauge modules. The system was designed so that there would be no single point of failure. In the event that one of the digital strain gauges failed it would not impact the ability to obtain data from the remaining. One additional channel was available to access the crackmeter data (Figure 3).
6. Power to the sensor nodes is controlled by the data logger component and are only powered while readings are being taken.
7. The data-logger batteries should last 180 days (D-Cell battery with 8.5Ah rating) with the logging schedule envisaged.
8. The 2Gb memory card can store nearly 4 years of data at a sample rate of 1 sample/10s.
9. Integrated Bluetooth wireless module with range up to 30 metres (line of sight).
10. Data can also be retrieved via a USB cable if necessary.
11. All equipment is sealed against moisture and corrosion.

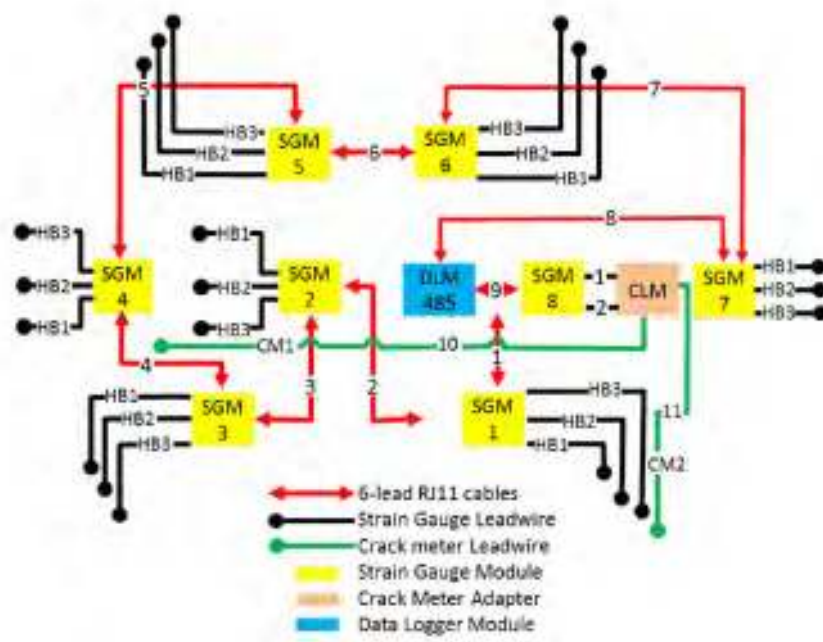


Figure 3: Segment cabling schematic

During the development stage a flexural test on a steel fibre reinforced concrete beam was performed to confirm that the proposed strain gauge solution would provide reliable results. The strain gauge module and three strain gauges were embedded inside the concrete beam and worked satisfactorily for the duration of the test (Figure 4).



Figure 4: SFRC beam flexural testing

2.2 Instrument location

The strain gauges (21 no. per segment) were located in the areas where the highest tension loads were anticipated (Figure 5) and two crack-metres were installed per segment on the intrados surface at location where cracks could develop.

Initially four segments of two SFRC lining rings and four segments of one conventionally reinforced lining ring were instrumented. However, two instrumented segments did not function when they arrived on site so ten instrumented segments were used for field testing and data collection.

The monitored segments were located on the upper half of the lining ring to allow easier access (Figure 8) and at locations where the rock was foreseen to be good so allowing more rigorous testing.

Figures 6a and 6b show the arrangement of the gauges relative to the thrust ram contact points on the ring. Three sets of three gauges were positioned to measure the circumferential strains in the ring. Two sets (SGM1 and SGM6) were arranged between the rams and one (SGM2) and a surface crack meter were arranged at segment midpoint directly in front of the thrust rams. These gauges were positioned at the middle of the segment thickness to avoid circumferential bending moments.

Four sets of three gauges SGM3, SGM4, SGM5 and SGM7 were positioned radially perpendicular to the circumference and in the centre of the ring.

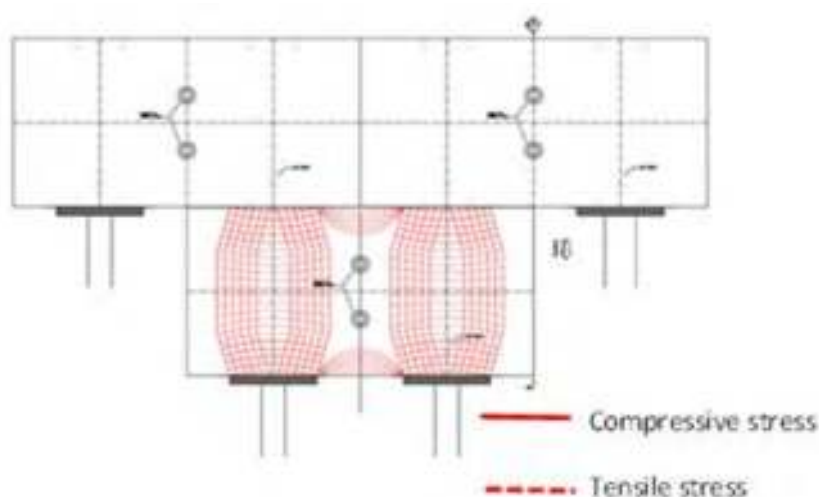


Figure 5: Expected principle stress trajectories

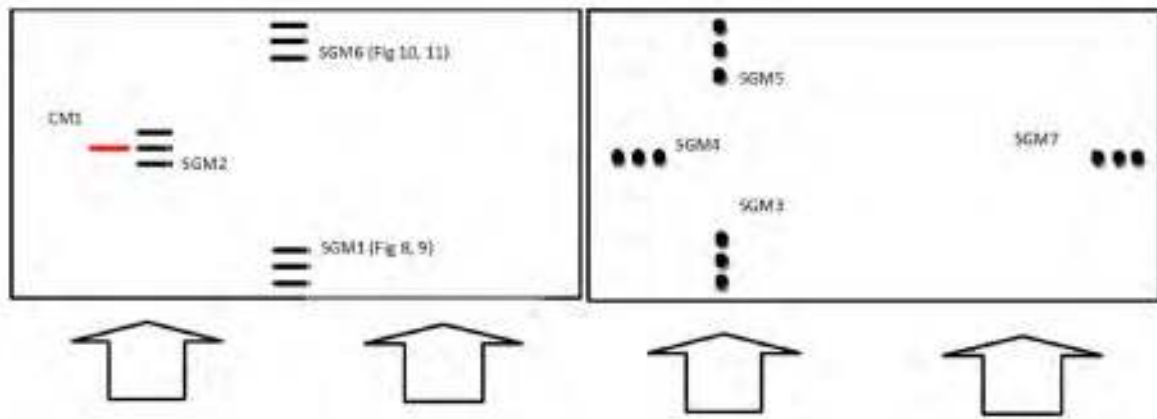


Figure 6a and b: Typical strain gauge arrangement a. Circumferential strains b. Radial strains.

3 INSTRUMENT INSTALLATION

3.1 Installation first stage

A first stage of installation was done at the segment precast yard, where all the strain gauges and strain gauge modules were embedded in concrete. The datalogger and the crackmeters were installed at the site in order to avoid damage during storage and transportation.

As the SFRC segments only contain a small amount of conventional reinforcement a frame was required to position and secure the strain gauges correctly and to train the wires to the position of the box out (for the data logger). The frames were constructed from 6mm diameter fibre glass bars which were tied with steel wires to the existing rebar (Figure 7).

The strain gauges were attached to the fibreglass GFRP rods with steel wires and plastic ties. The fibreglass bars have benefits over a steel frame as fibreglass has a similar elastic modulus to that of the concrete and so will not affect the behaviour of the SFRC around the strain gauges



Figure 7: Instruments mounted in-situ; 80% less wiring than VW methods

3.2 Instrument device

The monitoring instrumentation comprised:

- Seven groups of three strain gauges.
- Seven Strain Gauge Modules.

- Two cables to connect the crackmeters at the external surface and 2 cables to connect the equipment to the datalogger cast within the concrete.
- One box embedded into the concrete to create the recess where the datalogger will be installed (on site Liantang).

3.3 Installation sequence

The installation procedure was as follows:

- a. Crackmeter lead-wires were inserted within plastic pipes connected to the rebars with cable ties to protect them during concreting. Both ends were filled with silicone sealant. The part against the mould was protected with a sponge.
- b. Strain gauges were attached to the GFRP bars at the accurate locations using a spacer measuring tool. The position of each strain gauge was checked and adjusted to the theoretical position with a tolerance of $\pm 5\text{mm}$.
- c. The instrumentation was tested using a PC to check that all the components function correctly prior to concreting.
- d. The two cables from SGMs and the two crackmeter lead-wires were inserted into the plastic box-out which was filled with foam and wrapped with bubble wrap before concreting to allow easy extraction later.
- e. Concreting of the segment was performed with care in order to avoid direct dropping of concrete onto the strain gauges.
- f. Demoulding of the segment. Recovery of the crackmeter cables and cables in the plastic box-out.
- g. The instrumentation was again tested to check that all the components functioned correctly.
- h. The instrumented segments were marked on the leading and trailing edges and on the intrados for identification.

3.4 Lessons learned during installation at the Precast Yard

The environment in which tunnel lining segments are fabricated is harsh for electronics hence, all electronic components need to be designed for physical and thermal robustness and protected against water ingress.

The strain gauges need to be installed to a high degree of accuracy if the data is to be meaningful; less than $\pm 5\text{mm}$ which is less than tolerance usually achieved in a precast yard (for rebar placing for example). The strain gauges were eventually attached to the GFRP bars when the cage was inside the mould.

Placing concrete needs to be performed with care to avoid forceful dropping or flowing of concrete directly onto the instruments.

All cabling and electronic components need to be designed to resist vibration in a wet environment. The very strong vibration of the segment mould will affect cabling and electronic components if these are not effectively fixed in place.

Some leakage into the strain gauge module (SGM) occurred during the first trial due to a lack of sufficient waterproofing.

In order to mitigate this risk posed, the intensity of the vibration for these temporary segments was reduced. SGMs were suspended between rebar by wires instead of directly attaching them to the steel rebar to avoid excessive shaking during the vibration of the mould. The SGMs were filled with foam to make them completely waterproof.

3.5 Installation second stage

A second stage of installation was undertaken on site before segment erection. Here the crack meters were installed on the segment intrados surface, the instruments plugged to the data logger and data logger sealed in the recess in the segment. A Plexiglas plate was bolted on top of the data logger module in order to protect it from the erector vacuum pressure (around 1 bar). The crackmeter, adapter and crackmeter wires were covered with duct tape on the intrados surface to protect them from damage that could be caused by workers or the vacuum pad. A plastic cover plate was used in order to not impact the Bluetooth communication signal.

3.6 DLM installation details

Prior to DLM installation, the box-out for the DLM was removed. Care was taken to avoid damaging the cables which protruded from the concrete segment into the box and were required to remain intact for connection to the DLM. The data cables were then fed into the DLM through a hole with a rubber grommet located on the bottom of the DLM box.

Once all the data and crackmeter cables have been inserted and connected in the DLM, foam has been applied to the cable entry area and the lid screwed on top. The foam expanded in the confined space to form a protective barrier against the environment.

The unit was powered by 2x8.5Ah “C Cell sized” batteries. Once switched on, the DLM reads the internal configuration and starts operation. The DLM was tested to ensure that it was “reading” all available SGMs and data could be downloaded to the tablet.

Once the testing was complete the DLM lid was fixed and silicon applied around the edges to provide a water resistant barrier.

The last step of installation was to screw the Plexiglas plate on top of the DLM box. A foam gasket and silicone sealant were provided between the Plexiglas plate and concrete in order to provide an airtight and watertight system.

4 APPLICATION

The original firmware allowed for 10-second reading cycle. A tablet device with bespoke application was used to access and download files from the DLMs. The application included the ability to:

- Find all available DLMs,
- Check the DLM status,
- Change the DLM configuration settings,
- Download data files from the DLM,
- Modify the DLM polling schedule,
- Commission the DLM.

The ability to communicate with the data logger wirelessly was very useful to check that the DLMs were functioning properly and to change the configuration as necessary. Unfortunately, at Bluetooth baud rates data download times were lengthy which impacted on the efficiency of this system. It is recommended that future systems use real time data transfer over wireless mesh networks.

5 TEST PROTOCOL

The test protocol was predefined and included several parties: construction, survey and technical teams.

The load test itself consisted of increasing the jack pressure and maintaining the pressure at different steps. In order to more precisely control the pressure in the jacks the test was done in static mode, i.e. not in excavation mode and with the cutterhead not turning. To avoid damaging the cutterhead and disc cutters, due to high contact pressures with the ground, the cutterhead was retracted. The confinement pressure was increased in order to be able to reach high jack pressure values, up to 7.5 MN per pad.

TBM data records including thrust loads and jack elongation were obtained by the machines data logging system. Eccentricities and stepping between segments within an instrumented ring and between segments of the instrumented ring with the adjacent rings, were then surveyed and recorded.

Another important parameter to monitor was the development of cracks at the surface of the segments. After each step the instrumented segments were visually checked and cracks, if any, recorded and mapped.

The segment instrumentation logging schedule was set to the following;

- 1) First 2 days after erection: one reading every 10 seconds.
- 2) From day 3 to 15: one reading per minute.
- 3) From day 16 to 30: two readings per day.

6 OBSERVATIONS/CONCLUSIONS

6.1 Overview

Sensors were installed on four rings within the top hemisphere of the ring. One ring was conventionally reinforced and the other rings were built with steel fibre reinforcement. In this paper we shall consider one segment at the 10 o'clock position (Figure 7) and look at the circumferential strains between the rams; a location of tensile strains.

The monitoring device was designed to record the first thrust of the jack on the segment and avoid cable obstructing the working area.

The global analysis of the results shows the expected behaviour of the concrete at the location of the strain gauges (tension between and behind jacking pads both circumferentially and radially). This tension decreases rapidly towards the trailing edges which are predominantly in compression. Gauges at the trailing edge responded most to the grouting pressures.

In terms of cracks, segments with conventional reinforcement displayed more cracking (Figure 8b) than the SFRC segments with the same magnitude of load. This is reflected in the results where the peak tension strains recorded in the RC segment were four times those in the SFRC segment. The variation of strain with depth within the RC segment (Figure 9) was $1.5 \mu\text{strain}/\text{mm}$ whereas in the SFRC segment (Figure 10) the strain was more evenly distributed at only $0.33 \mu\text{strain}/\text{mm}$ and with lower tensile strain.



Figure 8a: Locations of instrumented segments with analysed segment shown



Figure 8b: Cracks on segment (TTR1172)

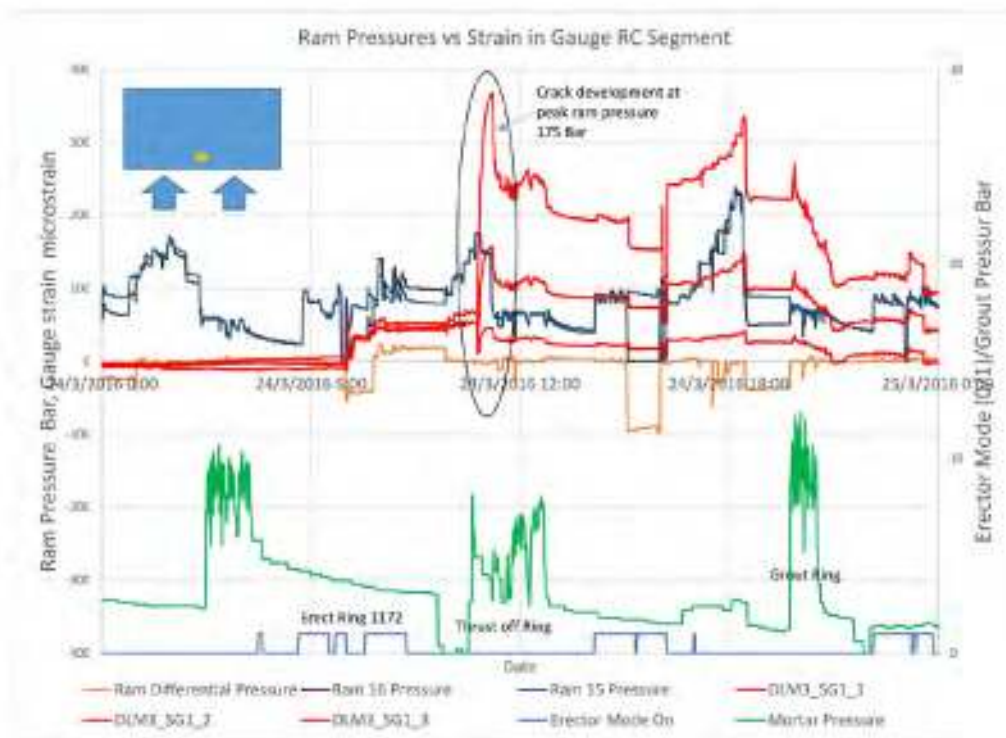


Figure 9: Stresses and Strains Observed on RC Segment Leading Edge (TTR1172)

6.2 Circumferential Strains

The profile in Figure 9 shows that in the early part of the thrust the tensile strain in the conventionally reinforced segment tracks that of the SFRC segment with tensile strain over applied stress (bar) at about 0.5 to 0.6. The lower values correspond to the conventional reinforcement. At peak thrust pressure there is a rapid increase in strain in the conventionally reinforced segment which reduces as load is removed but not down to levels expected for the thrust pressures being applied. The stress strain behaviour is indicative of concrete cracking. The data further shows the extent of the cracking with each gauge showing the same pre and post failure characteristics to different extents. The gauges were installed up to 300mm into the concrete and even the deepest sensor shows failure behavior indicating that cracks have propagated well into the reinforced area of the concrete. The cracks appear very sensitive to differential pressure between the two applied jacks with clear spikes in tensile strains when one or other jack was unloaded. This may have stimulated deeper propagation of the cracks. After the next

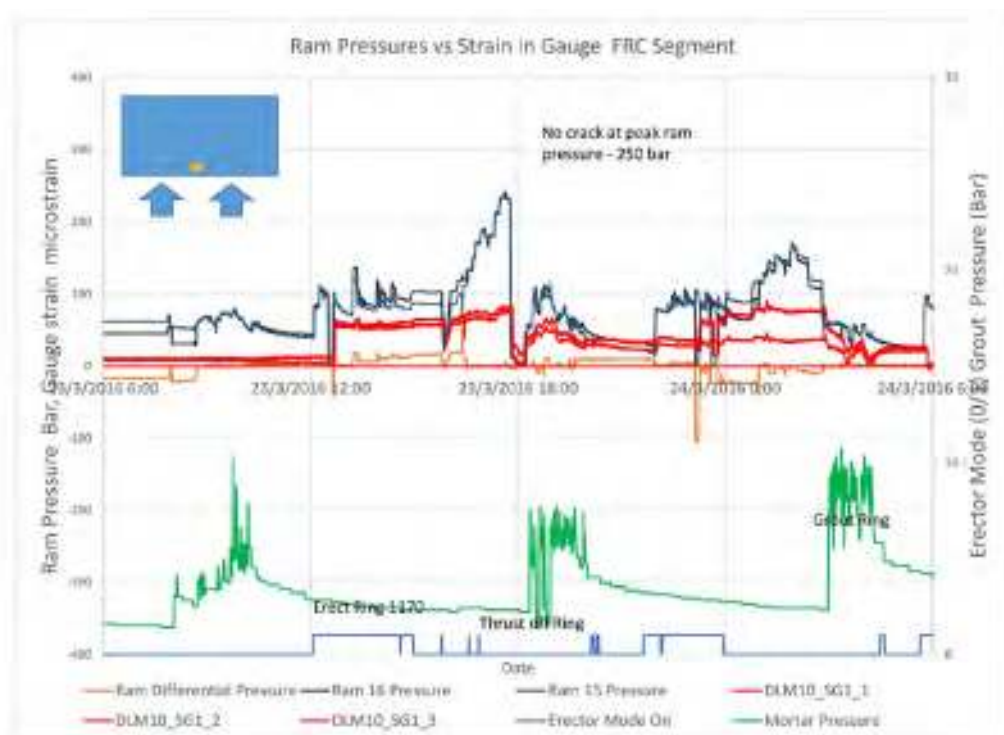


Figure 10: Stresses and Strains Observed on FRC Segment Leading Edge (TTR1170)

loading round on the subsequent ring, permanent concrete strains up to 100 μ strain persists indicating open cracking which would focus deeper corrosion.

The SFRC concrete (Figure 10) was subject to stresses 40% higher than the conventional concrete (240 bars) yet shows no such visible propagation of cracks. This highlights one of the main benefits of the SFRC approach; that of strain softening. In this case cracking is allowed to occur but as small micro-cracks at small strain none of which are pervasive. A plastic zone is then permitted to penetrate into the concrete which mobilizes more concrete capacity to withstand load.

Figures 11 and 12 show the distribution of circumferential strains near the trailing edge of the rings. At this location there is no tensile expression of the thrust loads in either the RC or SFRC segments only a small compression. Increased transient compression due to grout pressure can also be seen and dissipation of this pressure as grout penetrates into the surrounding ground is observed as soon as the grout pressures are removed. A slowly increasing trend can be seen over one day of observation which may be indicative of ground load transferring onto the ring.

6.3 Radial Strains

Radial strains in both RC and FRC segments are similar and track closely the application of ram thrust pressures and is shown in Figures 13 and 14. Generally radial strain is between 0.3 and 0.45 of the applied ram pressure in bars.

7 FUTURE CONSIDERATIONS

The data has provided valuable information on the dynamic behaviour of RC and SFRC segments during the installation processes. Future projects may benefit from attention to the following:

- Finding mounting solutions to avoid the use of additional GFRP bars to attach the strain gauges.
- Minimise as much as possible work to be done on site (i.e. install at the precast factory) and the interaction between workers and monitoring devices.
- Increase robustness of devices and cabling to avoid damage during installation and concreting.
- Increase the long term water resistance of all the monitoring devices.
- Employ remote wireless systems rather than local Bluetooth to check the status of the monitoring from a computer at the office.

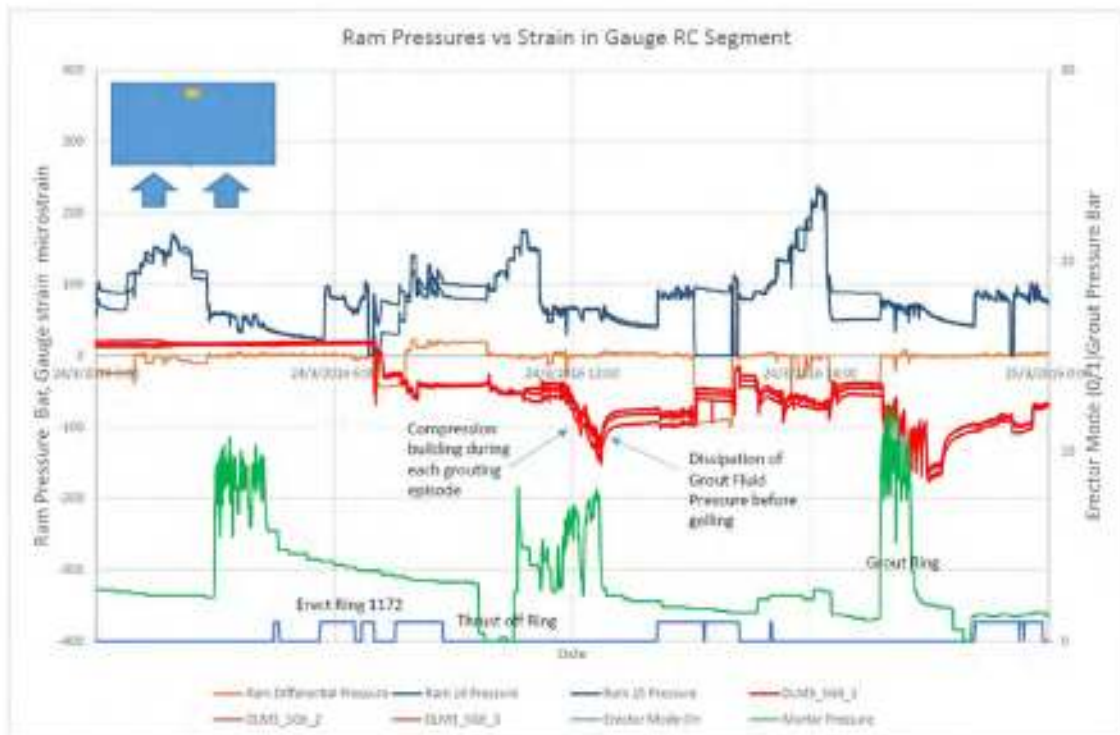


Figure 11: Stresses and Strains Observed on RC Segment Trailing Edge (TTR1172)

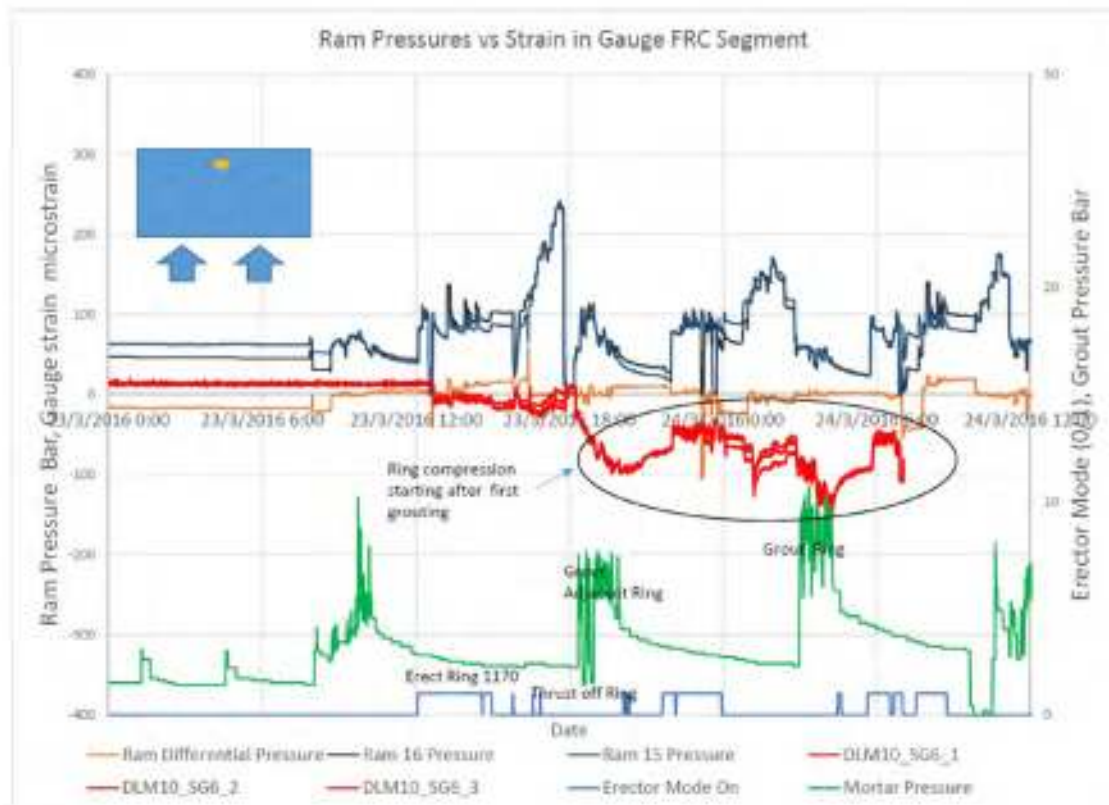


Figure 12: Stresses and Strains Observed on FRC Segment Leading Edge (TTR1170)

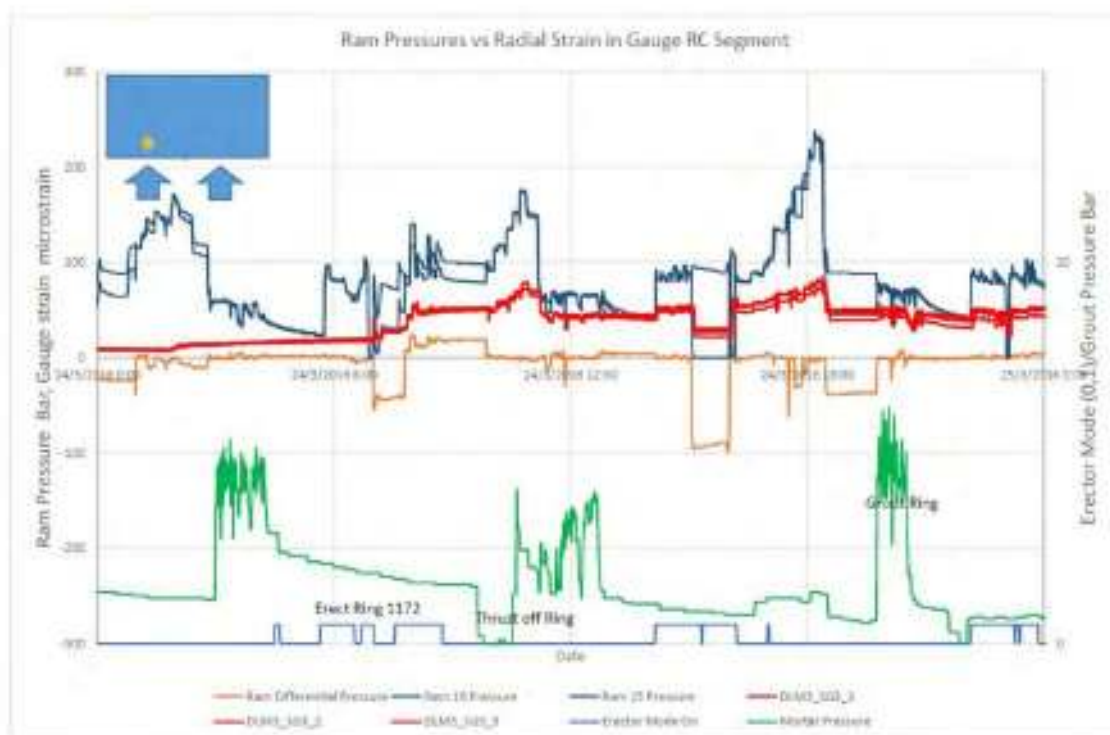


Figure 13: Radial Stresses and Strains Observed on RC Segment Leading Edge (TTR1172)

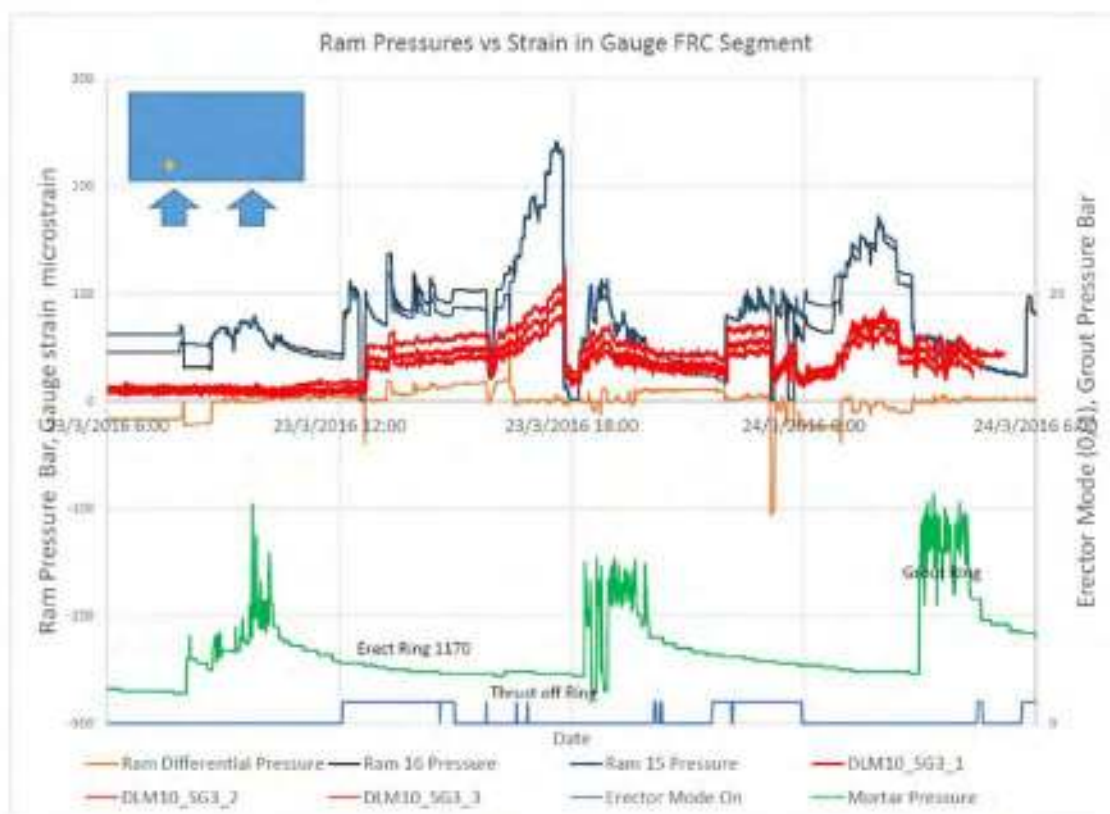


Figure 14: Radial Stresses and Strains Observed on FRC Segment Leading Edge (TTR1170)

Application of Advanced Software Programmes in a Mega-sized Railway Project Involving Substantial Embankment Construction

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ABSTRACT

In a mega-sized railway project in Southeast Asia, over 200km long of embankment is to be designed and constructed. The proposed railway alignment passes through terrains with variable altitudes and geology, from hilly areas with in-situ soils and rocks to low-lying areas overlying soft clays. Due to the limited time for design, advanced software programmes were used to analyse the geological and topographical data and to generate the geological profile, existing ground profile and proposed formation profile to facilitate the subsequent embankment design work. In addition, earthwork allocation plan was worked out and completed in short time with the aid of another advanced software programme.

In this paper, how the software programmes were applied to facilitate subsequent embankment design is presented. The strengths and limitations of the software programmes are also discussed.

1 INTRODUCTION

A mega-sized railway project was proposed in Southeast Asia and we were responsible for the design of two of the packages which included over 200km long of railway embankment. The proposed railway alignment would pass through terrains with variable altitudes and geology, from hilly areas with in-situ soils and rocks to low-lying areas overlying soft clays. According to the master programme of the project, all the design works including the detailed design drawings had to be completed within one year. As required by the client, cross sections had to be produced at an interval of maximum 25m along the alignment and around 8000 cross-section drawings had to be completed within a 2-month time. Furthermore, bills of quantities which included the detailed quantity of earthworks had to be completed at a similar deadline for costing purpose.

Due to the tight design programme, advanced software programmes “Hint CAD” and “Hint DZ” were used to analyse the topographical data and ground investigation data respectively. The software programmes then automatically generated the geological profile, existing ground profile and proposed formation profile which greatly sped up the production of the substantial amount of cross-section drawings. In addition, another advanced software programme “Hint TF” was used to work out the earthwork allocation plan, which allowed us to obtain the detailed earthworks quantities and facilitated the completion of the bills of quantities in short time.

Details of the application of the software programmes are given in the paragraphs below.

2 APPLICATION OF THE SOFTWARE PROGRAMMES

2.1 Generation of Ground and Geological Profile

Existing Ground Profile and Proposed Formation Profile

In the earlier stage of the project, LiDAR survey was carried out at the project area. The software programme “Hint CAD” allowed the use of the topographic maps containing 3-D data created from LiDAR survey to generate the existing ground profile. With this function, the project team not only produced the existing ground profile quickly, but was also able to identify locations which may be critical for design in an easier way by inspecting the 3-D topographical model generated by the software programme.

Only several steps are required to generate the existing ground profile. After importing the topographic maps to “Hint CAD”, the relevant layers in the maps, including spot levels and contours lines, were selected for generating a 3-D triangulated irregular network model (Figure 1). With the 3-D triangulated irregular network model, the existing ground profile of the project area would have been formed.

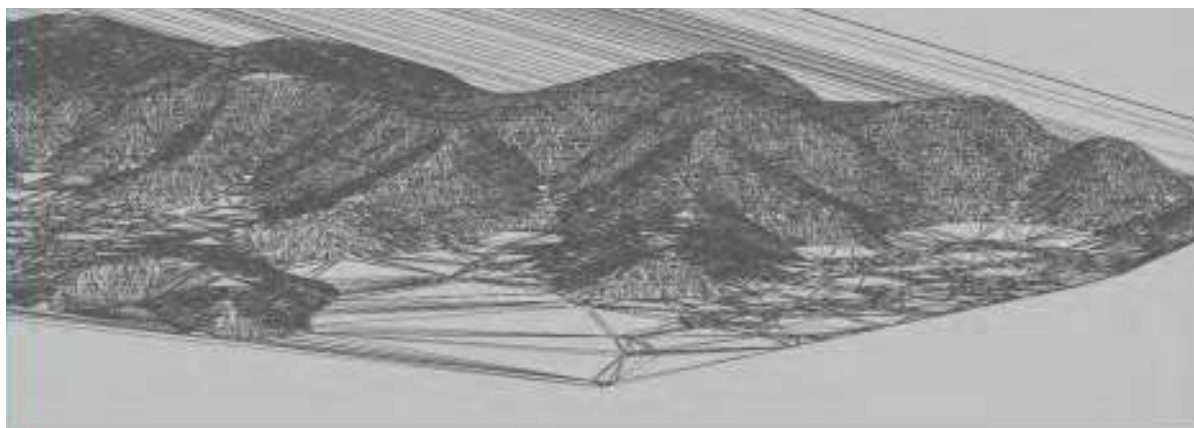


Figure 1: The triangulated irregular network model generated by “Hint CAD” for the project

For the proposed formation profile, i.e. the proposed embankment profile, the users were first required to define the “reference line” of the proposed railway alignment with elevations at different chainages along it in “Hint CAD” (in this project, the “reference line” is the centreline of the left track). The chainages and the corresponding elevations along the “reference line” were generated by “Mx Road” which is another software programme for alignment design. After that, the details of the proposed embankment, such as the width of the top of embankment, embankment fill slope gradient, cut slope gradient etc. were input through an input window in “Hint CAD” and the proposed embankment profile would then be saved by the software programme.

The whole process for generating the existing ground profile and defining the proposed formation profile was largely computerised with little manual input which minimised any possible human error.

Geological Profile

Another software programme “Hint DZ” was used to generate the geological profile of the proposed railway alignment. The data of the ground investigation drillholes that needed to be input included name of drillholes, location of the drillholes (in terms of chainage of the proposed railway alignment), offset distance from the “reference line” of the railway alignment and the details of each soil and rock layer of the drillholes (e.g. soil / rock name, decomposition grade, reduced level / depth etc.).

After all these data were input, the software programme would be able to generate the geological profile automatically, which could be a 2-D or 3-D geological model containing the drillhole data.

By combining the existing ground profile, proposed formation profile and geological profile mentioned above, longitudinal sections and cross sections at different chainages could be output as

we needed. Nevertheless, to produce presentable cross sections for detailed design drawings, post-processing of the cross sections by another computer-aid drafting software programme was required. For instance, the required ground treatment type and extent and some dimensions were added separately. Figure 2 shows a sample of cross section produced with all the existing ground profile, proposed formation profile and geological profile with the aid of the software programmes.

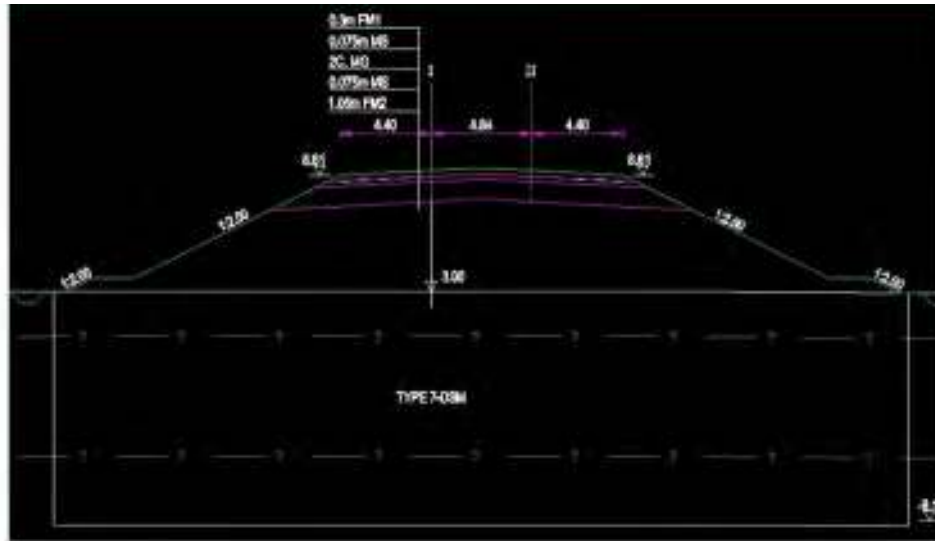


Figure 2: Cross section produced with the aid of the software programmes

2.2 Earthwork Allocation Plan

The earthworks involved for embankment construction in this project is significant due to the project size. To optimise the construction cost of earthworks, mass-haul shall be optimised by maximising short-distant earthmoving within the site and minimising long-distant material import and export. To achieve this, an earthwork allocation plan had to be worked out, which allowed the project team to clearly understand the cut and fill volume at each works area (defined by chainages along the proposed railway alignment), as well as how and where the surplus earthwork materials at a works area are to be transported to and deficient earthwork materials to be borrowed from.

In view of this, the software programme “Hint TF” was used which allows this earthwork allocation plan to be performed easily and fast with considering the enormous project extent and substantial volume of earthworks involved.

Before working on the earthwork allocation plan, several parameters were determined and input. These included the shrinkage factor for each type of soil, cut and fill volume at each works area, the proportions of different types of soils / rock at each works area, the type(s) of soil / rock which would not be considered in the “cut and fill calculations”, structures between the embankments (e.g. bridges and tunnels) and whether these structures would be allowed to be part of the earthwork transportation route, location of disposal pits and borrow pits, volume of different types of soil / rock to be disposed / borrowed at each disposal pit and borrow pit etc. (Figure 3 to Figure 5). Some of the parameters could be imported directly from the geological models generated as mentioned in Section 2.1 above which had saved much time for this step.

After inputting the required parameters, the next step would be the “pre-process of data” (Figure 6). In this step, the method of calculating the earthwork quantity, wastage percentage of earthwork materials in the course of earthmoving and how the earthwork materials are reused within the same works area had to be defined. This step serves for two purposes. On the one hand, this step works out the surplus and deficient materials at each works area after the earthwork materials are reused within the same works area; on the other hand, it counterchecks any error in the input data from the previous step.

With the above steps completed, the earthwork allocation plan was then carried out in a visualised graphical interface. The volume of each type of surplus and deficient soil / rock at each works area are



Figure 3: Inputting the proportions of different types of soils / rock at each works area



Figure 4: Inputting the information of structures between the embankments



Figure 5: Inputting the information of borrow pits and disposal pits



Figure 6: Window for "Pre-process of data"

presented in the form of vertical bars of various colours (Figure 7). Areas with surplus earthwork materials are represented by vertical bars above a horizontal axis while areas with deficient earthwork materials are represented by vertical bars below the same axis. After choosing the principle of earthmoving as per the project's needs, the earthwork allocation plan was simply done with the following steps:

1. Click the works area(s) that is to export surplus earthwork materials (Area "S1");
2. Click the works area(s) that is to receive the surplus earthwork materials (Area "R1");
3. The colour of the vertical bars at Area "S1" and Area "R1" will change, indicating that an earthmoving has been done;
4. Repeat Step 1 and Step 2 for the next round(s) of earthmoving until the whole earthwork is completed.

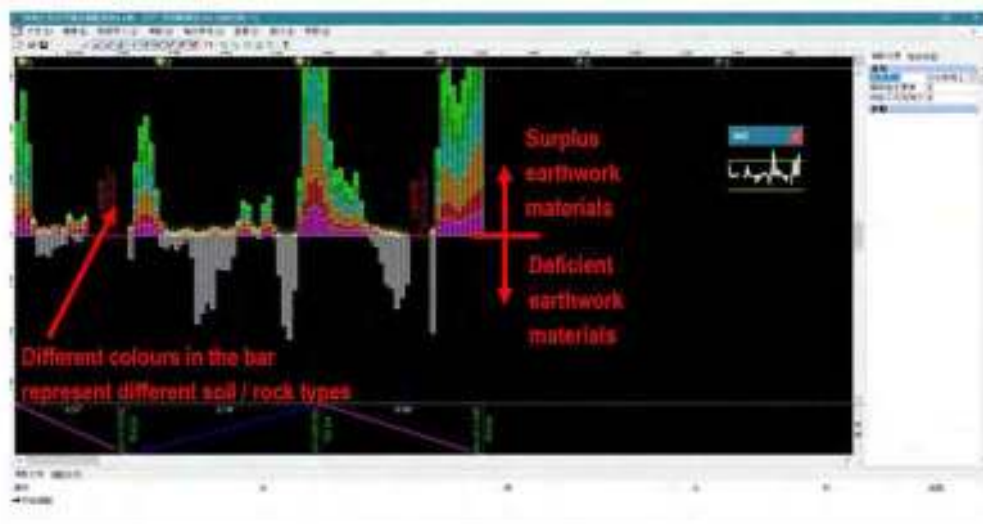


Figure 7: The user interface for the earthwork allocation plan

Upon the earthwork allocation plan was completed, various results were generated in the form of excel tables which eased reading and facilitated further analysing / processing of the data. For example, a summary of earthwork materials volume is shown in Figure 8. This summary tabulates the excavation volume of different soil and rock types and the fill volume required at each works area (in terms of chainage). It also tells the volume of excavation materials to be re-used within each works area, and the volume of earthwork materials that have to be imported and exported at each works area. Figure 9 is another example of summary table which shows the transportation distance of earthwork materials by different means at each works area. This summary table facilitates the planning of construction resources and estimation of construction costs relevant to earthmoving.

Chainage				Excavation volume of different soil / rock types														Fill volume (total / soils / rocks)				Volume of soils and rocks to be re-used within the works area / imported / exported							
工區	里程	土層	厚度	土層土質及數量 (m³)														填土數量 (m³)				利用土層及填土數量 (m³)							
				土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層	土層
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
K210+020	0.00	221.71	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+030	0.00	166.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+040	0.00	147.81	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+050	0.00	166.04	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+060	0.00	147.78	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+070	0.00	147.88	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+080	0.00	147.28	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+090	0.00	144.01	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+100	0.00	140.46	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+110	0.00	135.03	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+120	0.00	133.91	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+130	0.00	131.87	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+140	0.00	128.68	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+150	0.00	126.60	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+160	0.00	123.31	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+170	0.00	120.23	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+180	0.00	117.08	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+190	0.00	114.01	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+200	0.00	107.86	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+210	0.00	105.23	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+220	0.00	102.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+230	0.00	98.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+240	0.00	94.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+250	0.00	90.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+260	0.00	86.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00
K210+270	0.00	82.00	10.00	100	00	00	00	00	00	00	00	00	00	00	00	00	10000	10000	00	00	10000	00	00	00	00	00	00	00	00

Figure 8: Tabulated results of earthwork materials volume at each works area (土石方數量計算表)

站 区 段 号	土						石						备 注		
	人 工 挑 运		推 土 机 施 工		铲 运 机 施 工		挖 运 机 施 工		推 土 机 施 工		人 工 挑 运			自 卸 车 施 工	
	数 量	平 均 运 距	数 量	平 均 运 距	数 量	平 均 运 距	数 量	平 均 运 距	数 量	平 均 运 距	数 量	平 均 运 距		数 量	平 均 运 距
	(m³)	(km)	(m³)	(km)	(m³)	(km)	(m³)	(km)	(m³)	(km)	(m³)	(km)		(m³)	(km)
K218+000~K217+000	81.2	6.8					1282.3	8.181							弃 方
	128.9	12.3					348.3	12.101							弃 方
	125.4	0.3	2381.8	0.584											运 送 构 架
	2282.4	11.4					10464.7	12.371							弃 方
	409.4	11.2					7718.4	12.181							弃 方
K217+000~K216+000	1085.2	0.4					102108.3	0.434							弃 方
K216+000~K215+127.860	21408.7	8.3					444783.8	8.022							弃 方
K215+127.860~K212+900	3442.7	2.7					100181.4	2.693							弃 方
K212+910~K212+000	811.7	0.2			10182.8	0.188									运 送 构 架
K212+000~K212+070	211.8	1.8					4028.4	1.838							弃 方
	18.1	0.8	1.8	0.003											运 送 构 架
	9817.3	1.3					171181.3	1.330							弃 方
K212+070~K211+000	10.3	0.8	899.8	0.813											运 送 构 架
	3.8	1.3					44.3	1.078							弃 方
K211+000~K210+270	11.8	8.3					245.3	8.487							弃 方
	46.7	0.1	418.0	0.028											运 送 构 架
	849.8	8.4					8247.4	8.421							弃 方
	8469.7	8.3					122924.8	8.338							弃 方
K210+270~K208+178.200	12888.8	8.3					248883.8	8.328							弃 方
K208+178.200~K201+400	18188.8	9.3					148448.4	8.381							弃 方
合 计	100188.1		21117.8		10182.8		248883.8								

Figure 9: Tabulated results of the transportation distance of earthwork materials by different means at each works area (路基土石方运量统计表)

3 DISCUSSIONS

Because of the project requirements specified by the Client, the use of the various advanced software programmes has largely enhanced the efficiency for producing the ground and geological profiles (and hence the cross sections) and working out the earthwork allocation plan for this mega-sized railway project.

For the earthwork allocation plan, the software programme “Hint TF” in fact did not automatically do the allocation itself. It however did provide a user-friendly visualised graphical interface, which allowed users to carry out the allocation with simple manipulation, and finally the most optimised earthworks could be worked out. The user needed to complete the allocation based on their own experience and actual site conditions, with considering factors such as the cost of different transportation means and intended construction planning.

Nevertheless, some of the software programmes used are built based on the Chinese engineering standards and practice. For instance, the soil / rock types in “Hint TF” have been pre-defined according to the Chinese standard and users cannot define other soil / rock types. The language of “Hint CAD”, “Hint DZ” and “Hint TF” is basically in Chinese and the format of the ground and geological profiles also follow the common practice and style in China’s projects. These have limited the software programmes to be applied worldwide for international projects at the moment.

4 CONCLUSIONS

This paper presents a successful experience of applying various advanced software programmes in a mega-sized railway project involving substantial embankment construction in Southeast Asia. With the aid of the software programmes, the production of the ground and geological profiles, and thus the 8000 cross sections, for over 200km embankment of a proposed railway was largely computerised. Although post-processing of the cross sections directly generated from the software programmes was inevitable due to the level of details required for the embankment detailed design drawings, the use of the advanced software programmes has greatly sped up the overall drawing production and the task was completed successfully within a limited timeframe.

One of the software programmes used in the project, “Hint TF”, also provided a visualised graphical interface with simple manipulation, allowing the earthwork allocation plan, which involved more than

30,000,000m³ of earthwork volume over a length of 200km, to be completed with ease and quickly. Different tabulated results were also readily available which facilitated further analysing / processing of the data.

Early Insight into Subsurface Risk: A Digital Platform Approach

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ABSTRACT

Developers of critical infrastructure are often confronted with underperformance in delivering their projects on time and within budget. Failure rates are as high as 90% in the public sector and 71% for energy and natural resource projects. Subsurface conditions often are the largest factor of uncertainty in projects and significantly impact project economics and development time. However, design decisions are frequently made, in the absence of detailed insight into subsurface behavior. Consequently, the validation of design assumptions with data observed in the field is crucial, and can significantly reduce the construction risk profile. Digital platforms can be used to efficiently disseminate subsurface information to project stakeholders, improving the understanding of site characteristics early in the project cycle, as well as using visualisations and streaming analytics to provide insight into soil-structure interaction in near real-time. Such an approach can offer significant benefits for both project stakeholders and society if applied responsibly. This paper explains how digital platforms may be used to mitigate ground risk based on a hundred projects conducted. It shares lessons learned illustrated by two recent projects undertaken: the construction of the Victory Boogie Woogie tunnel and levee reconstructions with real time risk control.

1 INTRODUCTION

1.1 Underperformance in the construction industry and subsurface failure costs

Commercial and programme underperformance is common in the construction industry. KPMG (2015) notes that more than half of all construction projects are not completed on time and under budget. In the public sector and the energy sector, even higher failure rates are seen, at 90 percent and 71 percent respectively. Overall, less than one-third of the respondents' projects surveyed by KPMG managed to come within 10 percent of the planned budget, with the public sector performing especially poorly. KPMG states that one of the biggest concerns is the accuracy of the anticipated costs estimation prior to committing to the project. Projects are moving so fast that project teams have limited time to formulate the scope and accurately estimate the costs. Almost 80 percent of the engineering and construction companies surveyed believes that project risks are increasing, and that application of commonly used contingency rates (10 percent) do not adequately cover the project risks. Contingency planning typically involves downside risk estimates for budget and delivery times throughout the project life cycle.

Subsurface conditions have a significant impact on developing time and budget. According to Gupta (1998), insufficient site investigation can increase the final construction cost by up to 25 percent. Fookes et al. (2000), Knill (2001), and many other sources indicate that failure to anticipate unforeseen ground

conditions is one of the largest elements of technical and financial risk in civil engineering projects. A priori, the value of subsurface intelligence is the highest, while geotechnical data are often not available. During the feasibility or front-end engineering design stages, stakeholders and engineering teams have to make key (investment) decisions without sufficient insight into subsurface behaviour. Estimation of contingency, plot layout selection are often conducted without sufficient understanding of the site and associated risk. The role of engineering geologists in risk assessments is most effective during these initial phases of a project, as well as the value of subsurface intelligence.

2 TRADITIONAL APPROACH TO GEOTECHNICAL INFORMATION AND GROUND RISK

2.1 Data availability

Traditionally, it can take months before a detailed site investigation can be planned and procured during the front-end engineering and design process, and further time is needed for the execution, which typically will also be of the order of months. Before a final Geotechnical Baseline Report is issued, engineering teams would typically have spent typically months producing designs and plot layouts, with only limited subsurface information available. Uncertainty related to the subsurface and ground risks is almost impossible to quantify responsibly without a comprehensive overview of reliable geotechnical field data and documented information on the local geology. Analysis of the site investigation results is mostly centred around hard-copy reports containing drill logs, SPTs, CPTs and laboratory testing results, and it is almost impossible to develop a practical integrated data-driven or modelling approach to contribute to the overall understanding of the site.

2.2 Traditional reporting and digital accessibility

Although the Geotechnical Baseline Report is an invaluable tool for the Geotechnical professional, there are some disadvantages to the way that traditional hard-copy reports limited the accessibility of critical subsurface data for interpretation purposes. Physical reports restrict the availability of information to a limited number of stakeholders. Distribution of critical information takes time, and further analysis of the acquired data is frequently limited as it is often only available in printed or photographic representations. Traditional reports often only provide one-dimensional information per research point, omitting relevant geospatial context. Geotechnical profiles often help to enhance understanding of stratigraphy, but can only provide a two-dimensional interpretation, whereas good geotechnical design requires a three-dimensional approach. The appreciation and use of models that provide a three-dimensional representation of data (see figure 1), as well as digital delivery and digital data sharing practices is becoming increasingly valued in the industry (Patton 2016; Baru et al., 2008; Kennedy, 2017), as well as beginning to be required or encouraged by regulators around the world, including Hong Kong (Buildings Dept, 2016).

2.3 Validation of design assumptions and modern ground risk management techniques

Risk registers can help geotechnical engineers to quantify the value and effectiveness of site investigations, as can probabilistic techniques such as fuzzy fault-tree analysis and Markov Chains

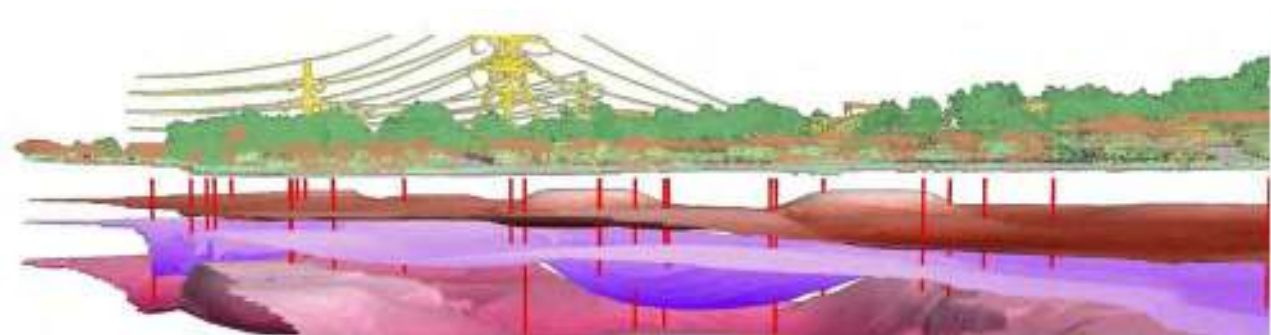


Figure 1: 3D visualization above ground (high density point cloud) and below ground (model), source: Fugro

applied to spatial uncertainty of soil behaviour. Such methods have already been successfully applied (Jaksa et al., 2003), but the early involvement of geotechnical engineers is still required to help quantify ground risk in terms of project commercial outcome, in order to be able to make effective trade-offs during the initial design stages of a project. Before construction commences, the uncertainty related to ground behaviour frequently remains large, and in particular there is often limited insight into the dynamic component of ground behaviour. To mitigate this, monitoring, and dynamic soil-structure interaction, is becoming increasingly important.

For optimum benefits to be gained, this approach does require a change in philosophy from that commonly encountered whereby the specifications for monitoring campaigns appear to be driven by the number of sensors, rather than designing them based on identified risks and geospatial context. Furthermore, the value of real-time automated telemetry systems to facilitate early warnings and reduce overall monitoring cost are often overlooked when designing monitoring campaigns, with the availability of fully integrated real-time project data remaining a dream in three out of four projects (KPMG, 2016).

The use of monitoring, augmented by technologies such as outlier detection and cloud processing, can deliver immense value during the construction process by providing validation of the design, prevention of non-brittle geotechnical failure and early warnings of potential issues. If such monitoring campaigns are implemented in conjunction with pre-planned mitigation measures, they can enable real-time risk control (Pot et al., 2015). Fugro's digital platform approach has been applied in over a hundred projects worldwide, in both land and marine environments and has adequately processed and visualized more than 2 billion geotechnical measurements. This approach has led to an increase of data use: efficient exposure of site characteristics to project teams and relevant stakeholders generally is, on a project basis 10-100 times higher when using digital interfaces.

3 CASE STUDY 1: ROTTERDAMSEBAAN VICTORY BOOGIE WOOGIE TUNNEL

3.1 Project Characteristics

The Rotterdamsebaan is a 4 km section of two-lane, dual carriageway road currently under construction in the Netherlands, that will connect the A4 and A13 highways to the main ring road surrounding The Hague (Rotterdamsebaan, 2018a). The alignment of the new road section passes through heavily built-up urban areas, and a major part of the alignment is underground. The underground section is 1.9 kilometres long and contains the Victory Boogie Woogie Tunnel (named after the final work of Dutch abstract painter Piet Mondriaan, held in the collection of the Hague Municipal Museum). The Victory Boogie Woogie Tunnel comprises two bored tunnels, each 1.6 kilometres long with an internal diameter of approximately 10 metres, with connections between the two tunnels at 250 metre intervals. The deepest point of the tunnels is 31 metres below the surface. The tunnels are constructed using a Tunnel Boring Machine (TBM).



Figure 2: (left) Tunnel Boring Machine (courtesy of Herrenknecht); (right) the Rotterdamsebaan Tunnel Boring Machine (courtesy of Municipality of The Hague)

The TBM (Figure 2) consists of a cutting wheel, excavation chamber and thrust system, and the complete system is around 80 metres long, with a cutting wheel of approximately 11.3 metres diameter. The cutting wheel is rotated to remove soil while being pushed against the soil face by the thrusting system acting against previously installed concrete tunnel lining segments. When the jacks of the thrusting system are fully extended, a new lining segment is installed using a moveable robotic arm. Once the new segment is installed, the jacks are retracted to act against the new segment, and the process is repeated. To limit soil displacement and surface settlements, grout is injected behind the tunnel lining segments.

3.2 The subsurface and ground risk

The TBM operates at a relatively shallow depth, between eight metres and 31 metres below the surface. The subsurface has been mapped with almost 400 Cone Penetration Tests and sufficient boring logs to validate the soil classifications from the cone tests. This has resulted in a detailed geotechnical length profile (Figure 3). To the east (right side of the profile), the top soil layer consists of soft, cohesive material, predominantly peat (green). Between three metres and ten to 14 metres below the surface, there is a silty sand layer (yellow), sometimes interrupted by a clay film. Below this, there is a two metre thick clay layer (brown/grey) covering the sandy aquifer (orange) with cone resistance generally above 10 MPa. Despite this shallow traverse, the maximum allowed surface settlements are typically only 12 mm. Settlement predictions anticipate a trough-like shape perpendicular to the tunnel alignment (Figure 4). Depending on their distance from the tunnels, ground conditions and the foundations of adjacent structures, more than 300 assets in the vicinity of the tunnel alignment, including business and housing, are exposed to varying levels of risk associated with differential settlements (rotation/sagging and hogging) caused by the tunnelling process.

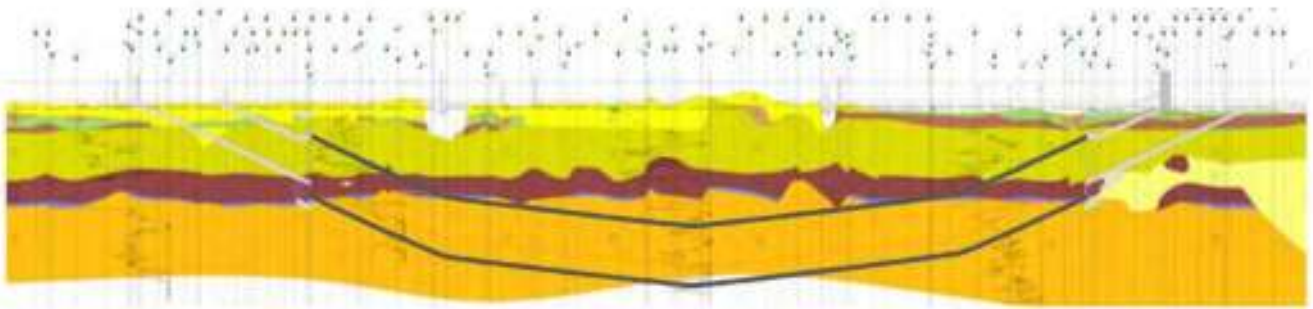


Figure 3: Geotechnical Length Profile – Victory Boogie Woogie Tunnel

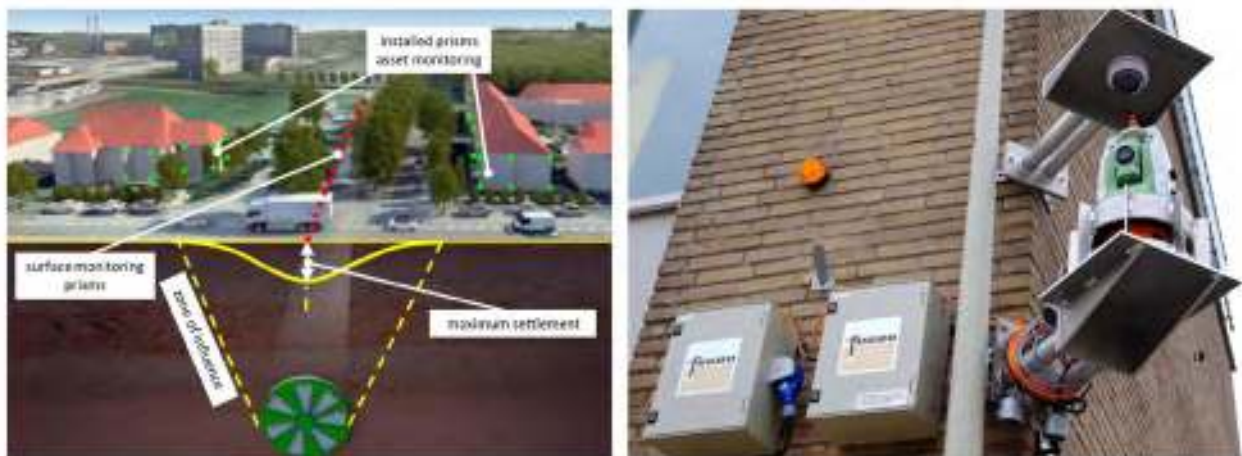


Figure 4: Schematics of the surface (left, source: Municipality of The Hague) and a robotic total station (right)

3.3 Objective of the monitoring

Because of the significant risk of ground settlement and low allowable settlement limits, an extensive monitoring campaign has been put in place to optimise TBM operations (reducing potential surface settlement), to control the risk of damage to adjacent assets by providing continuous real-time insight. A third reason is to validate the observed against the set settlement requirements. Together, these objectives allow the Municipality of The Hague, the project owner, to demonstrate to the public that they are fully in control of the construction process, helping to gain and maintain public support for the project. For this reason, the Municipality of The Hague chooses to execute the monitoring independently of the tunnel contractor in a separate scope of work.

3.4 Monitoring campaign aimed at real time control

Fugro put in place an extensive monitoring campaign based on geodetic control measurements (relative and absolute x, y and z in mm). To collect the field data, 38 Robotic Total Stations (RTS) with automatic levelling and 6,000 prisms were installed. An estimated 200 million data points will be collected over the life of the project, with monitoring of the surface and assets adjacent to the tunnel alignment on a 24/7 basis. The monitoring campaign includes an initial baseline monitoring phase, as well as a manual levelling campaign to provide a check on the automated system. At the TBM entry shaft, three lines of prisms have been installed perpendicular to the alignment to validate the predicted influence on ground settlement.

The data management system provides near real-time insight, allowing the contractor and independent project office to control ground risk and tunnelling operations (see figure 5). The system is part of Fugro's Gaia Insight platform, and uses cloud computing and MOVE3 for continuous adjustments of the designed network of Robotic total stations; as well as ingredient outlier detection technology to provide an early warning notification service. Using colour code visualisations and an interactive timeline, the vast amount of data is aggregated, providing actionable intelligence on a dashboard to non-technical end users. Users can access every measurement in graphs, to correlate measurements with the TBM position, cross sections and the effects on the 300 assets being protected. At the individual asset level, rotation, sagging and hogging movements can be verified to be within the tolerable levels set during the design and pre-construction stage.



Figure 5: Overview of the Gaia Insight Decision Support Platform: the start shaft tunnel, the dotted lines indicate the predicted surface settlements, and the red, yellow and green icons indicate prism locations and surface settlement.

The system provides input to TBM operations with more than forty graph configurations available, insight into the status of assets and visualisations of the predicted zone of influence, as well as several hundred views of the facades of the assets being monitored. In addition, all subsurface data, the live TBM position and metadata are all available to users, and an interactive timeline on the dashboard allows users to review historical performance.

Figure 6 provides an example of some of the near real-time surface settlement data acquired when the TBM passed the monitored zone. Each of the six time-series plots represents a different surface settlement point, at locations approximately 20, 40, 60, 80, 100 and 120 meters from the TBM entry shaft. The figure demonstrates the reduction of surface settlement (z-direction) as the TBM progresses over the first 120 meters, and convergence of surface settlements towards displacement of the

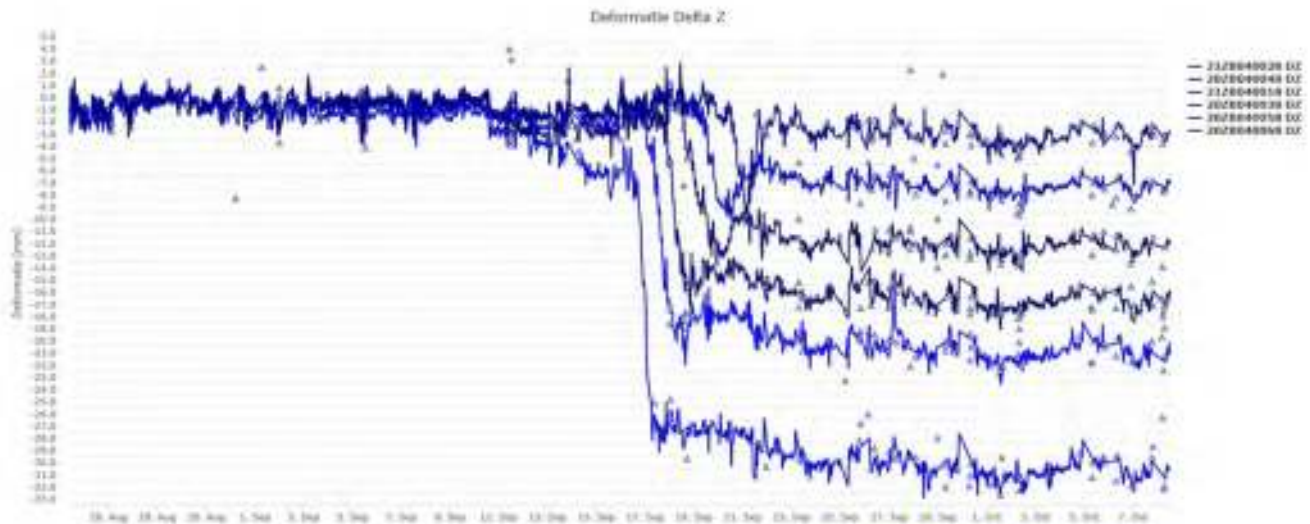


Figure 6: Relative surface settlements in mm over time

surface the order of a few millimetres. The time series consist of validated measurements and outliers (presented as small triangles). Along the planned tunnel alignment, a total of 800 surface points per tunnel was installed, corresponding with a monitoring point every 2 meters to closely record the surface settlements. Near the TBM entry and exit shafts, roughly 1,200 prisms, reflector-less data points and virtual sensors provided insight into the influence of the excavations on the adjacent environment. The system enables access to all measurements acquired, from raw data to aggregated results.

3.6 Lessons learned and benefits

The colour-coded real time visualisation of the monitoring points allowed users to quickly gain insight into the surface settlement. Outlier detection helped to significantly reduce false notifications. The use of cloud computing reduced latency, even when accessing large data sets. Uptime of the digital platform, which was well above 99 percent, is not a key factor for the monitoring campaign from a technical perspective, however it is a key prerequisite for the project owner to be able to demonstrate towards to key stakeholders and the broader public that they are in full control of the construction process. To avoid unnecessary downtime, maintenance of the monitoring scheme, connectivity, and hardware, and redundancy of the scheme, as well as on-call technicians require attention during the planning stage. To guarantee uptime, key performance indicators for data availability were set and the availability of measurements was optimised. The project manager stated: “The digital platform approach and Fugro’s monitoring scheme allows employees of the project organisation, as well as employees of the contractor to have real-time insight into the influence on the adjacent environment as a result of the tunnelling operations. Literally with a single glance, we can check if the contractor is in control.” (Rotterdamsebaan, 2018b). Over a hundred staff members of the project organisation (owner and contractor), as well as other key stakeholders were given access to the system during operations, contributing to the overall public support.

4 CASE STUDY 2: DIGITAL GROUND RISK MITIGATION IN DIKE RECONSTRUCTIONS

4.1 Project Characteristics

This case study describes a reconstruction project for a primary flood defense scheme in The Netherlands. Under the National Flood Protection Programme (HWBP), the regional authorities and Rijkswaterstaat (the Directorate General for Public Works and Water Management), are required to ensure that the 3,767 kilometres of dikes, dams and dunes forming the primary defences against flooding in The Netherlands comply with legal safety requirements. If, during a regular inspection and assessment, flood defences

are found not to comply with the safety standards, reconstruction works to strengthen the dikes are required by law. The assessment process includes checks against critical failure mechanisms.

The water authorities covering the Holland Delta area of the south-west Netherlands have planned a reconstruction scheme for 8.3 kilometres of dikes between the towns of Goudswaard and Oud-Beijerland. These dikes, along the River Spui, have to withstand large hydraulic loads as the river is the inter-tidal connection between two different parts of the Rhine Delta. The safety assessment for these dikes indicated that embankment erosion was a potential cause for liquefaction of some sections, the same geotechnical failure mechanism that led to the 1953 flood disaster in The Netherlands, resulting in 1,836 fatalities. The reconstruction and strengthening activities were carefully planned by the stakeholders, making use of several site investigations as well as earlier safety assessments. This led to a scope reduction, a better understanding of the dike system and resulted in a flood prevention plan that would allow reconstruction works to continue throughout the storm season.

4.2 Scope of the Reconstruction Works

The first phase of the reconstruction started in 2014 with the strengthening of the underwater slope with rubble and rock, to stabilise the underwater slopes and prevent erosion. The second phase of the reconstruction targeted the dike structure, which had a design life of fifty years. The second phase work was intended to improve macro stability by reshaping the dike dimensions, excavating a new drainage ditch, improving of the clay cover layers and the heightening of the soil core (see figure 7). In some areas, sheet pile walls were driven in the ground to enhance overall stability and decrease potential seepage during storm conditions.



Figure 7: Four cross sections for reconstruction, orange is new preliminary dike design (courtesy Water Authorities of the Holland Delta)

4.3 Ground risk management using bow-tie technique and digital platform

To improve communication relating to ground risk in the project, a digital platform approach was formulated. Historic data was made available early in the design process and a decision support system (DSS) was implemented during the reconstruction works. The system enabled access to all relevant data, including, but not limited to, all site investigation data, real time monitoring and the annotation of field observations. To formulate a more systematic approach, the bowtie technique was applied to evaluate the geotechnical risks to the dikes (Santosa, 2017). Bowtie diagrams were originally developed by the oil industry as a method of evaluating HSE risk. The technique takes its name from the shape of the diagram, which looks like a man's bowtie. The Bowtie diagram provides a visual summary of root causes resulting in an unwanted event, together with the consequences of the event such as financial damage, delay and reputational damage (Figure 8). In addition to a risk register, digital risk annotations were applied as a key feature of the DSS to interpret monitoring and to communicate potential unwanted events geospatially.

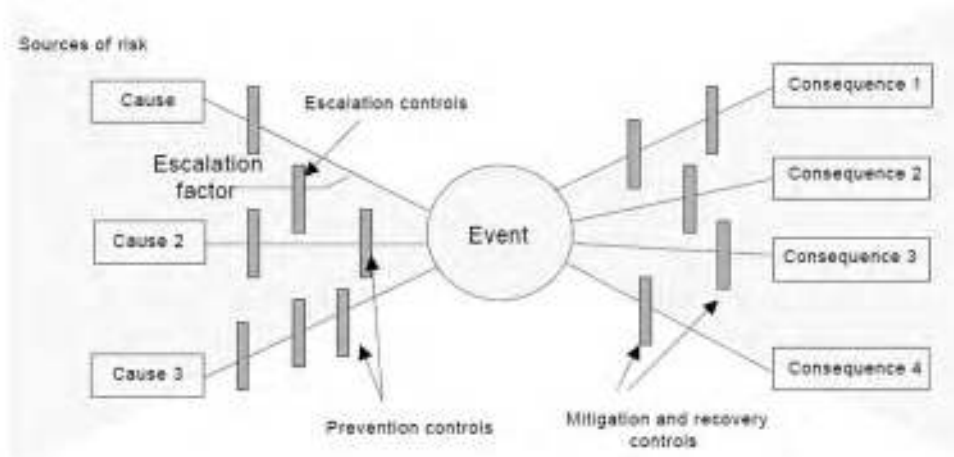


Figure 8: Concept of the Bow-Tie analysis (source: ISO31000)

4.4 Monitoring scheme and ground risk management approach

The scope of the second phase comprised earth works (embankment phasing) and the installation of sheet pile walls. To monitor excess pore pressure and consolidation, approximately 60 pore pressure meters and roughly 50 standpipes were installed. All data acquired was sent via a telemetry system to the platform, making the data immediately available to all relevant stakeholders. After a sensitivity analysis, threshold values were set for the different dike sections. With a short scenario description, the DSS was used to control the logistics of the earth works, consequently optimising the speed of the embankment phasing in a safe manner. The DSS (Figure 9) provided insight into overall stability of the inner and outer slopes using Bishop slip circles and visualised the derived status of the section in real time. When the section is green, the new top load could be applied. As high river levels could impact the validity of derived scenarios, real-time water level monitoring was added to the system. Using the monitoring campaign, identified risk and field observations, the system provided insight in the status of inner and outer slope stability, sensitivity of adjacent assets to the foreseen construction works, potential damage as a result of installation of sheet pile walls, seepage and flooding. Liquefaction risk and embankment erosion were previously mapped and covered in Phase one of the reconstruction works.



Figure 9: Gaia Insight – excess pore pressure as indicator for reduced slope stability during construction

4.6 Lessons learnt and benefits

A workshop to align the project organisation was critical to the implementation. The identification of wet clay layers in the field and effective communication of this, caused users to carefully follow the decision support system without undue cautiousness during embankment staging of each section. It is critical for users to be able to systematically annotate field observations. As the as-built dimensions of the embankment staging might differ slightly from the design dimensions, the resulting uncertainty should be taken into account. No system, that claims to cover everything, or every data source, can replace visual field observations and effective risk communication. The overall application of the system shortened the project by three months, helped to mitigate ground risk (avoiding geotechnical failure) and enhanced trust of the end client. No significant geotechnical failure occurred at the sections that the decision support system covered. The application of the bow-tie method and ground risk annotations contributed to effective risk communication.

5 TECHNOLOGICAL ADVANCEMENTS

In the early stages of a project, the analysis of open data sources and archive data contributes to better scope site investigations and design. Digital delivery of geotechnical site investigation data can support project teams significantly as they have the ability to access all available data from the beginning of the project.

A digital platform approach makes it easier to unlock historical geotechnical site investigation data and visualise it together with current data in a single representation. This enables rapid (even real-time) data sharing and supports a collaborative working method to turn this data into actionable intelligence. This is one of the cornerstones of Fugro's Digital Foundation, a digitised, spatial and geo-data framework that provides a four-dimensional model that provides users a deeper understanding of their site and their assets. It enables, amongst other aspects, (near) real-time connectivity of the geotechnical data acquisition and design validation to mitigate ground risk. It utilises ingredient technologies such as machine learning and streaming analytics, aimed to help making better design decisions earlier in the project cycle. The real-time component enables quicker adjustments during construction if needed. A digital platform approach can significantly increase the exposure of geotechnical data and subsurface intelligence to users, allowing more professionals, adjacent domains and related parties to contribute to mitigate ground risk. Its application continues to require strong involvement of geotechnical engineers and engineering geologists to utilize the technology responsibly.

6 CONCLUSIONS

This paper has demonstrated the use of a digital platform (Gaia Insight) to mitigate ground risk in real time. The approach enhances interaction with site investigation data, field observations from monitoring campaigns and decision support. This contributes to shorten project cycles and improve ground risk mitigation. The authors consider that a digital approach can benefit society when applied responsibly and at the same time will contribute to the bottom-line result of a project.

ACKNOWLEDGEMENTS

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Spatial Data Infrastructure of Ground Investigation Information

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ABSTRACT

In Hong Kong, geotechnical data of ground investigation, laboratory test, etc., in the form of hard copies and soft copies have been used in the past 40 years. AGS (Association of Geotechnical and Geoenvironmental Specialists) version 3.1 format data has been introduced since 1990s. The implementation of AGS format data is pioneering change in handling the available geotechnical data of the projects but the tools to handle such large amount of data are to be further explored so as to effectively and efficiently analyse and manage the data of existing and future projects.

Digitalised geotechnical data in the format of AGS files are available for the ground investigation (GI) works and laboratory tests. To centralise and store these AGS data, an online spatial data infrastructure (SDI) was developed for managing the available geotechnical data. The SDI is a data infrastructure implementing a framework of geographic data, metadata, users and tools that are interactively connected to use spatial data in an efficient and flexible way. It is a system that collects, catalogues and organises all such data in an organised way. Furthermore, once the data is centrally stored, analysis can be carried out effectively using database querying techniques. Currently the SDI could manage AGS data of Hong Kong, Macau, UK, Singapore, and Australia on a trial base.

1 INTRODUCTION

To develop ground model and obtain geotechnical parameters, geotechnical data of ground investigation (GI) and laboratory tests are necessary and usually obtained for the projects. The data is available in different formats - in AGS format (a digital, database ready format) and GI data available in PDF format.

A research has been conducted by Arup to explore and design a digital platform to manage extensive (but unorganised) collection of digital GI data (both AGS and PDF). In view of the enormous amount of GI data and their values to develop ground model, the digital platform should be able to collect, catalogue and organise such GI digital data to be used in an effective way. Furthermore, once the data is centrally stored, analysis can be carried out effectively using database querying techniques.

Therefore, a spatial data infrastructure (SDI) is proposed and preliminarily developed by Arup for managing geotechnical data for Hong Kong in first phase, and other countries using AGS data to store geotechnical information. The SDI is a data infrastructure implementing a framework of geographic data, metadata, users and tools that are interactively connected to use spatial data in an efficient and flexible way.

2 BACKGROUND

The ground investigation (GI) information contains all factual data related to the ground conditions such as material types, strata depth, results of in-situ testing, groundwater monitoring results, laboratory

testing results, and etc. Such GI information are presented in digital format which can be obtained and used by different parties. In view of this need, the Association of Geotechnical Specialists (AGS) established a standard electronic format by the publication of “Electronic Transfer of Geotechnical Data from Ground Investigation”, which is currently well-known as AGS Format (Craig, 1996). The AGS is an interchange format which allows transfer of data within the geotechnical industry. The use of AGS data has helped the contractor and designer to avoid the need to repetitive input the ground investigation information and aid the interpretation more efficiently (Craig, 1996).

Given the AGS data becomes a standard requirement for most projects in Hong Kong since 1992, an extensive GI database has been built up in Hong Kong region by Geotechnical Engineering Office (GEO) of Civil Engineering and Development Department (CEDD) (formally known as Geotechnical Control Office). The “Geotechnical Information Unit” (GIU) System of Civil Engineering Library houses a comprehensive collection of these GI data throughout the territory of Hong Kong and the GI data can be viewed online in Geotechnical Information Infrastructure (Ginfo). The GIU System is a significant development in GI data management so that the user can search relevant information spatially. The storage of AGS data online is on project based and may contains GI data in a large area.

In view of these, a new database, called Spatial Data Infrastructure (SDI) has been developed and aimed to manage the vast amount of GI data in digital format. The first phase of SDI uses available GI data in Hong Kong as pilot study. The SDI houses two major type of digital format: AGS and PDF. For the AGS data, the SDI allow import, process and export functions; for PDF file, the SDI allows “drag and drop” function. All digital files will be presented on the SDI website and allow users to download according to their specific purpose.

The PDF files will be displayed on the website with the site location indicated spatially. The AGS files imported to the SDI will be processed and the details are given in Section 5 below. The main purposes to process the AGS files are two folds: 1. to check the integrity of the AGS file; and 2. to ‘dissect’ the project AGS file to AGS file contain only one GI station, in order to display the GI station individually.

The SDI has been designed for community contribution in long term such that all holders of GI data, in either format, could contribute to the SDI in a user-friendly method and thus a comprehensive SDI containing GI data is expected. The SDI has been set with a function of administrator check, such that only data with accurate and consistent format and data with known ownership are allowed to be imported into the SDI to ensure data integrity.

3 FRAMEWORK

The work-flow of the SDI is shown in Figure 1.

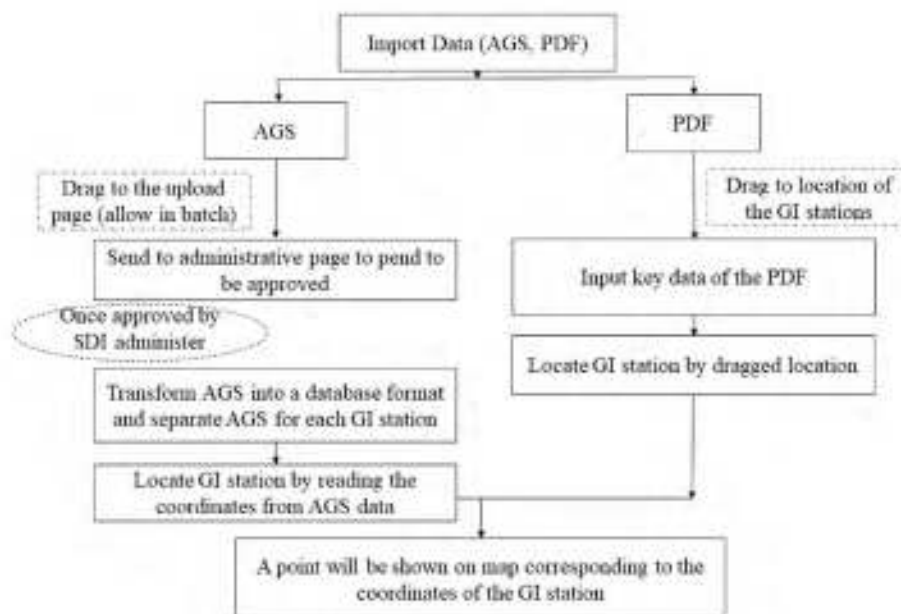


Figure 1: Framework of the SDI

4 IMPORT DATA

4.1 AGS data

dataset can be used as an input data to be stored in the geospatial database. It is known that AGS Version 4 will be used and the SDI will also be updated to incorporate the version upgrade of AGS format

4.2 PDF file

The AGS data format, as part of GI factual data, was implemented widely in Hong Kong only since 1990s. For GI data prior to this time, where AGS data may not be available, the PDF file may be the only source of GI data source. The SDI could use PDF files as input data to upload to the geospatial database to indicates the available GI location. In addition to GI reports, geotechnical reports and laboratory testing reports could also be uploaded to the database with geo-referencing, for user to search relevant geotechnical information.

5 PROCESSES

5.1 Methodology

5.2 AGS data

The SDI stored the AGS data in three parts: 1. in a Web Server, 2. SQL Server (SQL database) and 3. GIS Server (GIS data). The system comprises of backend data transfer to SQL Server and front end data transfer to GIS server to update the GIS features. The front end codes is written in Javascript, whereas the backend is written in C# language.

In general, the AGS data are stored in SQL Server tables. The tables structures is developed to be consistent with the structure of each AGS tables (e.g.: PROJ, HOLE, PTIM, etc). In addition, GIS Server also stores the spatially data such as: the borehole locations, borehole ID, and Project ID, which are taken from the AGS data, table: HOLE - HOLE_NATE, HOLE - HOLE_NATN, HOLE - HOLE_ID, PROJ - PROJ_ID, respectively.

When the users drag the AGS files to the SDI website, the files will be read by javascript codes, which subsequently will be uploaded and copied to a web server. After the uploaded data are approved by the SDI administrator, the AGS file contents will then be extracted by the C# codes and inserted into SQL database. Subsequently, the codes trigger another javascript codes to store the data into GIS server to create new GIS features, as points on the map, and show the GI stations on the website spatially.

The AGS file can be dragged into the browser and will be uploaded to the web server (Figure 2). Batch uploading is also available and coordinates system (e.g. Hong Kong, Macau, Singapore, UK, and etc) has to be selected before the upload. The upload process includes:

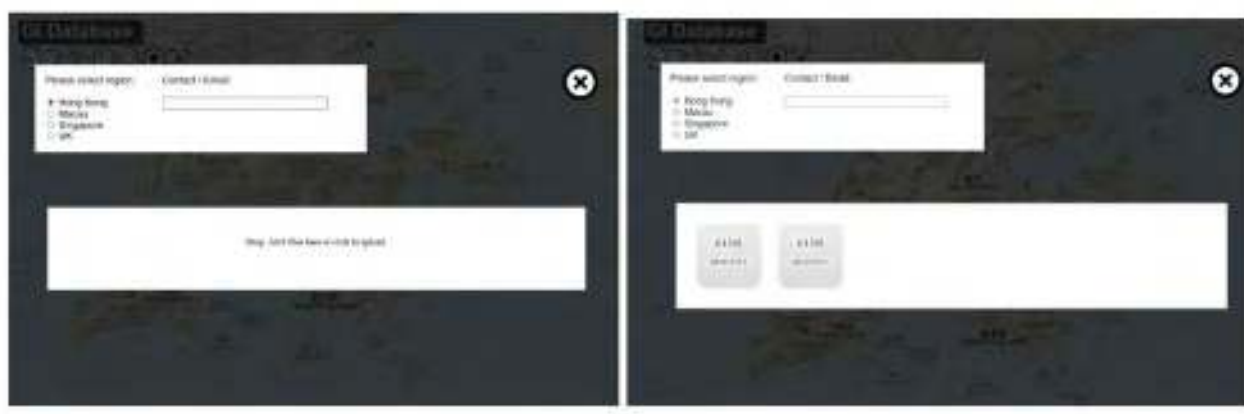


Figure 2: Upload page of the SDI

- Dissect the AGS data which may include numerous GI stations, into individual GI stations
- Transform the AGS data of GI stations into a standard database format (Figure 3Error! Reference source not found.)
- Read the database to get the coordinate system of a GI station

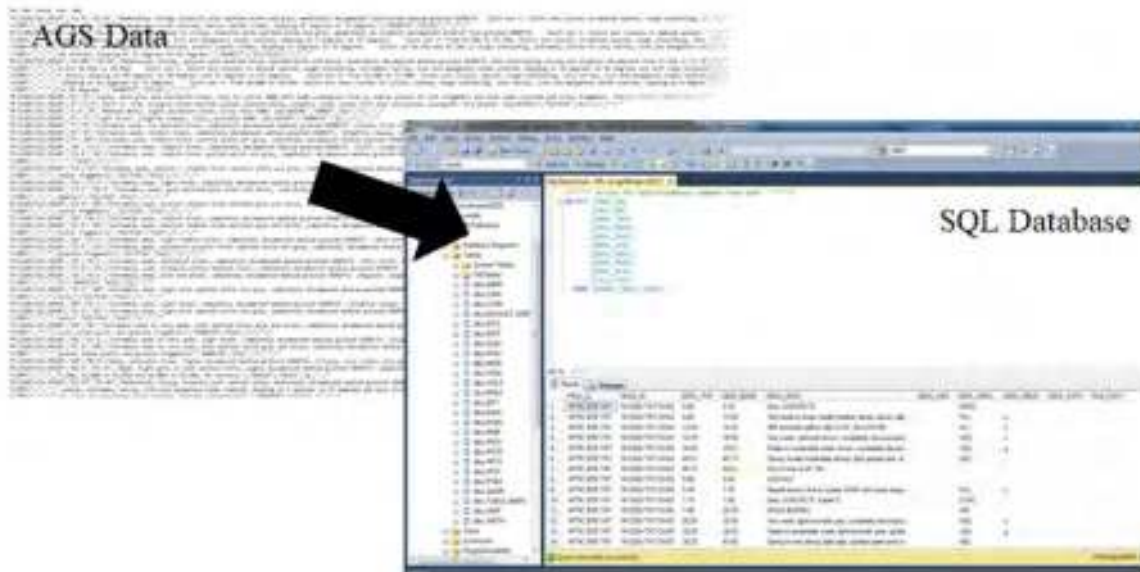


Figure 3: The AGS data is transformed saved into SQL database

5.3 PDF data

On the other hand, GI PDFs upload have few differences from the AGS upload. The PDF upload does not require SQL server to store the data. All user inputs are stored in the attribute table of the GIS data. Also different from AGSs upload, the PDFs upload requires a location input from the user. The location will be marked from the location the user drops the PDF file on the map.

The rationale of the GI PDFs upload are: 1. the users drag and drop the PDF file on the map; 2. the system, using javascript, requires user to input necessary information, such as: project identifier, contact person, and remarks; 3. the users submit the uploaded file, a point will be created on the GIS layer on the map, and all information will be stored inside the attribute table of that new point. 4. the PDF file will also be copied to the web server, and the link to that file on the web server is also stored in the attribute table for the users to access the file.

In addition, users can also drag a PDF to an existing GI point on the map to add an attachment to that individual point. Users can do it by selecting the point first prior dragging and dropping.

The PDF file will be stored in the web server. The key things of this upload include:

- Drag the relevant PDF report on the project location directly.
- Fill in basic details like project identifier, ownership, contact person, any remarks for usage, etc (Figure 4).
- upon uploading, a link of the report will pop up associated with the project point at the map.

In the map viewer, PDF data and AGS data will be presented as separated layers with different symbology.

6 ADMINISTRATOR PAGE

The aim of the SDI has been designed for peer contribution in order to populate the GI data as extensive as possible. However, in order to ensure the data integrity and the copyright details for each file are indicated, a control point is introduced. An administrator page has been set up so that all the imported data will be reviewed and verified before populating into the SDI and shown on the map.



Figure 4: View of SDI showing GI Stations of AGS and project location of PDF

7 IMPLICATION AND WAY FORWARD OF SDI

Digital GI data has been produced, stored and used in Hong Kong since 1990s and are considered one of the most extensive GI databases in the world. The development of SDI focuses on storing and managing the GI data more effectively. It aims to allow rapid and efficient, and centralised storage and retrieval of these available information and reduce the time in gathering such data, and minimize the abortive works such as duplicate inputting (sometimes error inputting) effort whilst allowing a more formal QA procedure to be followed.

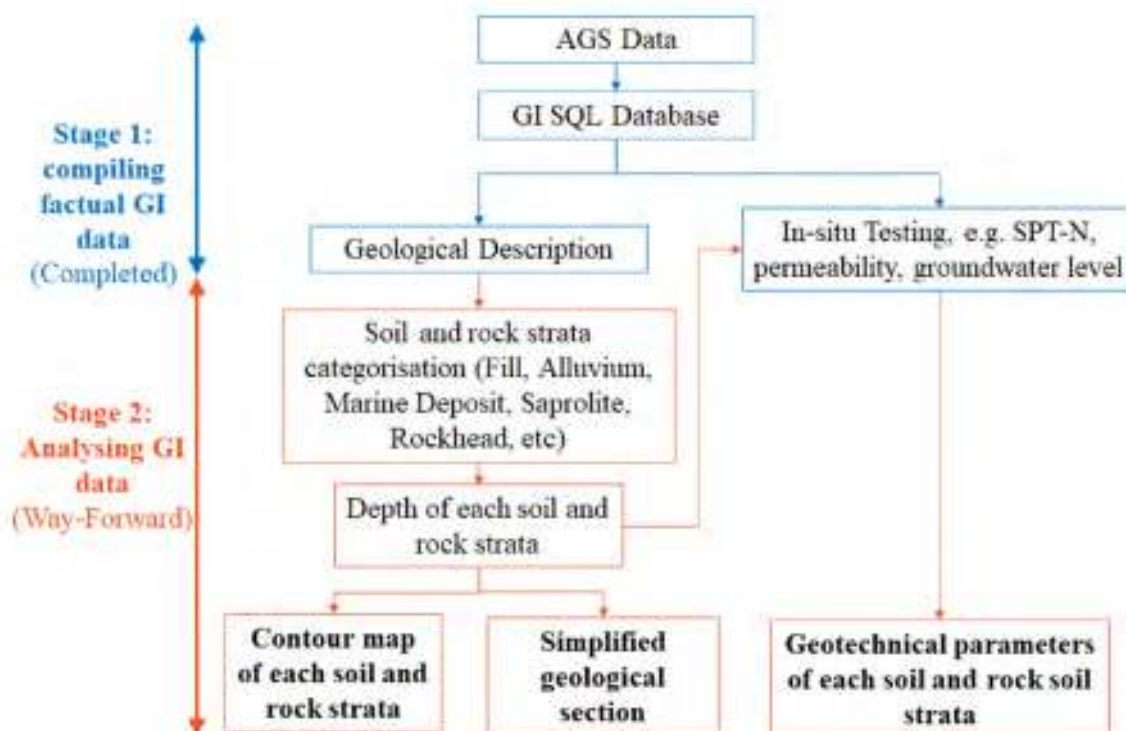


Figure 5: Flowchart of next stage of SDI

In addition to data storage and management, the SDI could also make use of the AGS file, which is factual data, to useful information particularly in ground modelling and ground characterisation. The stored AGS data in SQL server table format could be further used and allow the analysis of the subsurface ground condition in different aspects such as defining major soil strata and rockhead level, groundwater level, soil and rock parameter and etc for each GI station. Subsequently, the analysis of the multiple GI stations could generate a citywide scale of ground models for planning and engineering purpose. Analysing the GI data and produce geotechnical charts, tables, graphs, contour and 2D/3D ground models will be next stage of the SDI development (Figure 5).

8 CONCLUSIONS

As the GI stations can be imported from AGS files and presented spatially on map, this will facilitate the storage and retrieval of relevant geotechnical information for any projects more effectively and efficiently. The development of SDI also facilitates further analysis of the GI data in effective ways so as to understand the subsurface ground conditions in a defined study areas, or even an area if a comprehensive GI database is available, stored and managed wisely.

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Physical Modelling Study of a Debris Flow Impact on a Flexible Barrier

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ABSTRACT

Understanding the impact mechanism of a debris flow is of great necessity for the analysis and design of debris flow-resisting flexible barriers. In order to investigate the dynamic response of a flexible barrier impacted by a debris flow, a physical modelling experiment was performed by releasing a muddy debris flow to impact on a flexible barrier in a short period of time (within 1 second). The impact characteristics and the loading distribution on the flexible barrier during the impact process are investigated. Based on the observations and findings from the experiment, a novel simple method is proposed in this paper for calculating the dynamic impact force of a debris flow on a flexible barrier. The new simple method is validated by comparing the calculated values with the measured dynamic impact force in the experiment.

1 INTRODUCTION

Flexible barriers were firstly commercially developed by BRUGG in the 1970s to halt large boulders (Kane et al. 1993; Hearn et al. 1995) and suggested by Hong Kong administrative department in the report (Halcrow China Limited 2011). Recently, researchers have found that flexible barriers are effective in trapping debris flows (Canelli et al. 2012; Wendeler et al. 2007; Cui et al. 2015; Hu et al. 2006; Kwan et al. 2014). In the past years, flexible barriers have been sporadically hit by debris flows, shallow landslides and snow avalanches. From back-analysis of those cases, flexible barriers performed well in stopping and retaining major components of geophysical flows (Kwan et al. 2014 ; Margreth and Roth 2008). Some flexible barrier manufacturers such as Geobrugg modified the flexible rockfall barriers to improve the efficiency in debris flow mitigation.

Comparing with traditional countermeasures such as check dams, flexible barriers have four obvious advantages: First of all, the flexible barrier system is light in weight, which can be installed in the headstream of a risky valley with the aid of helicopters to mitigate debris flows in the early stage (Wendeler et al. 2007). Secondly, major components such as the flexible ring net and the energy dissipation elements can be replaced after being damaged, and the trapped debris can be removed by uninstalling the net (Volkwein 2014). Thirdly, capability of large deformation and permeability of the flexible barrier can attenuate the impact force and increase the capacity of trapping debris flows. Specifically, the large deformation of the flexible barrier can lengthen the impact period and decrease the impact loading peak, and the permeable secondary mesh net allows slurry and small particles pass through the flexible barrier with residual velocities to reduce the impact force.

In Hong Kong, brief guidelines and suggestions were developed by the Geotechnical Engineering Office, CEDD (Kwan and Cheung 2012; Sze et al. 2018) in conjunction with various academic and practicing groups (Ng. et al. 2003; Yung and Ng. 2017; Koo et al. 2016). In the design of flexible barriers

for debris flow mitigation, accurate determination of the impact force is the key issue. Nevertheless, the understanding of the interaction characteristics of a debris flow with a flexible barrier, especially for accurately determining the impact force during the interaction, has not been fully investigated. The objectives of this paper are investigating the impact characteristics of a debris flow on a flexible barrier by physical modelling experiments and developing the existing simple method for calculating the dynamic impact force in the design of debris flow-resisting flexible barriers.

2 EXPERIMENT SETUP AND PROCEDURES

2.1 Experiment setup and instrumentation

A physical modelling facility has been built in the Road Lab of the Hong Kong Polytechnic University. This facility contains three main pieces of apparatus (see Fig.1(a)): (1) a steel tank with a capacity of 5 m³ to store mixed debris material; (2) a chute with a length of 7 m, a width of 1.5 m and an inclination of 35 degrees; (3) an instrumented flexible barrier located 5 m below the releasing door of the storage tank.

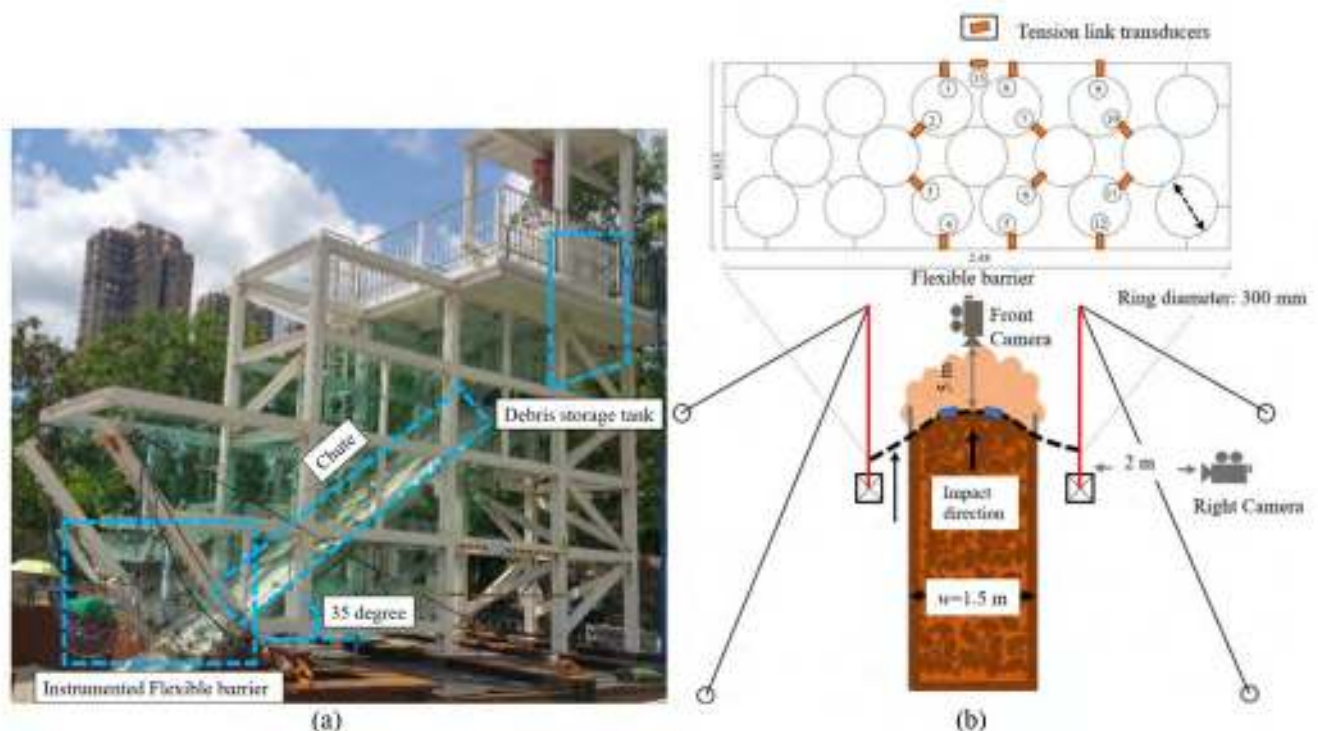


Figure 1: (a) Photograph and (b) instrumentation of the physical model facility for debris flow tests

The flexible barrier utilized in this study is commonly used for debris flow mitigation in Hong Kong, which consists of two layers of net: a primary ring net with the ring diameter of 300 mm (No. ROCCO 7/3/300, Geobrugg) to sustain major impact forces and intercept large boulders and a secondary mesh net with the mesh size of 50 mm to trap smaller particles and allow slurry pass through. The instrumentation of the facility is presented in Fig.1(b). Two high-speed cameras (No. MotionBLITZ EoSens Cube6) with the frame rate of 1000 fps in the resolution of 1024*768 were located in the right side and in front of the flexible barrier to capture the impact characteristics of the debris flow and the dynamic response of the flexible barrier. Thirteen mini-tension link transducers with the calibrated capacity of 20 kN were installed between adjacent rings of the flexible ring net to measure the tensile force directly on the primary ring net. The signals of the transducers were recorded by a dynamic datalogger (No. NI PXIe-1082). Using the maximum tensile forces of the mini transducers, the maximum impact force on the flexible barrier with n transducers, F_{measured} , can be calculated using the following equation:

$$F_{\text{measured}} = \cos \frac{\theta}{2} \cdot \sum_{i=1}^{i=n} F_{\text{tensile},i} \quad (1)$$

where $F_{\text{tensile},i}$ is the maximum tensile force on Transducer i installed on the flexible barrier; F_{measured} is the maximum impact force in the measured area; and θ is the included angle of the deformed measured area. The instrumentation and the impact force measuring method have been introduced detailedly in Tan et al. (2018a) and Tan et al. (2018b).

2.2 Experiment material and procedures

Debris flow in this study is made up of a mixture of Completely Decomposed Granite (CDG), aggregate and water. The basic parameters of the debris flow in the experiment are plotted in Table 1. The PSD (Particle Size Distribution) curve and the composition of the testing material are plotted in Fig.2. Once the experiment was prepared, the trap door was flipped up in less than 0.5 s by a self-invented mechanical fast release system to form a debris flow. To prevent the leakage problem in the preparation stage, highly compressible rubber bars and silicon glue were sandwiched into the gap between the locked door and the reservoir. Then, the debris flow moved down through the chute and impacted on the flexible barrier. Before the impact, the high-speed cameras started to record the whole impact process, and the datalogger started to document the signals of the transducers on the flexible barrier.

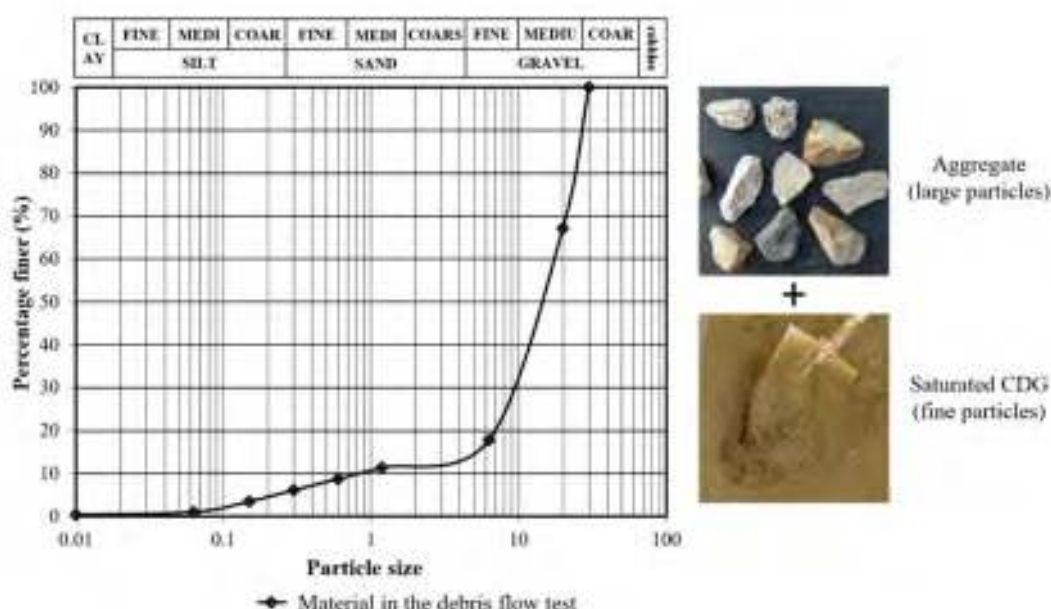


Figure 2: PSD and composition of the debris material used in the debris flow test

3 DEBRIS FLOW IMPACT TEST

In this study, a debris flow with the total volume of 2.05 m³ was initiated to impact a short flexible barrier (0.915-m-high) with a small retaining capacity (around 0.94 m³). During the interaction, the flexible barrier was impacted and filled by the debris flow in a short period (less than 1 second). The impact process has been recorded by the photographs of the side-view and the front-view high-speed cameras in the frame rate of 200 fps. Typical photographs have been selected to reveal the main impact characteristics (see Fig.3). Initially, when a debris flow impacted on an empty flexible barrier, around 70% of small particles and slurry passed through the mesh net of the flexible barrier with a residual velocity, which led to the consideration that only partial momentum of the passing slurry transformed into the impulse on the flexible barrier. 30% of the debris mixture, mainly large particles and debris, were trapped and filled in the flexible barrier, which blocked the mesh net and prevented continual passing-through of the debris flow.

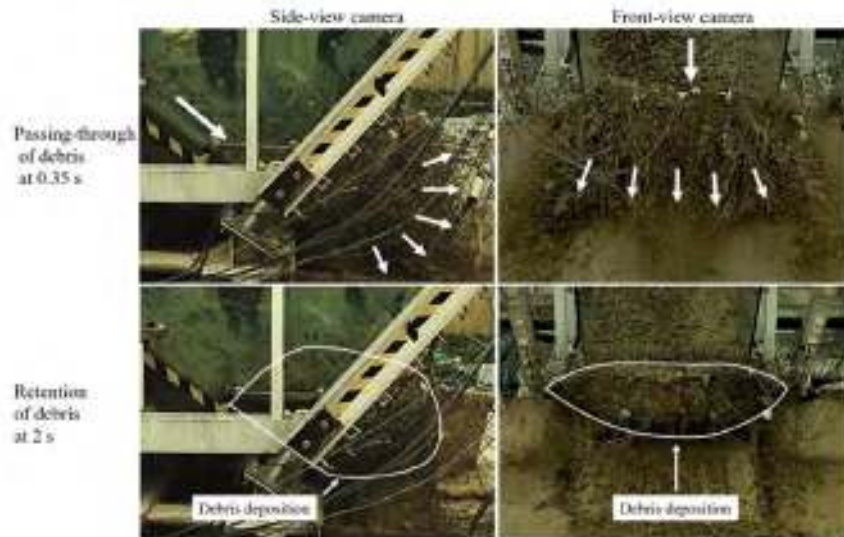


Figure 3: Typical photographs of the debris flow test representing passing-through and retention of debris flow

To better understand the impact loading distribution on the flexible barrier, thirteen mini-tension link transducers were installed on the flexible barrier. The arrangement of the transducers is plotted in Fig.1(b). The tensile force histories of all mini-tension link transducers in the debris flow test are presented in Appendix A. By selecting typical time points based on the force history of Transducer 13, the tensile force distributions on the flexible barrier at different typical times are shown by the contours in Fig.4. From those contours, it can be observed that the debris flow firstly impacted on the bottom central area of the flexible barrier at the time of 0.255 s. With accumulation of the debris deposition, the impact area of the dynamic loading on the flexible barrier moved up and reached the maximum value in 0.350 s. After the impact, the static loading of the debris deposition mainly acted on the upper central area of the flexible barrier.

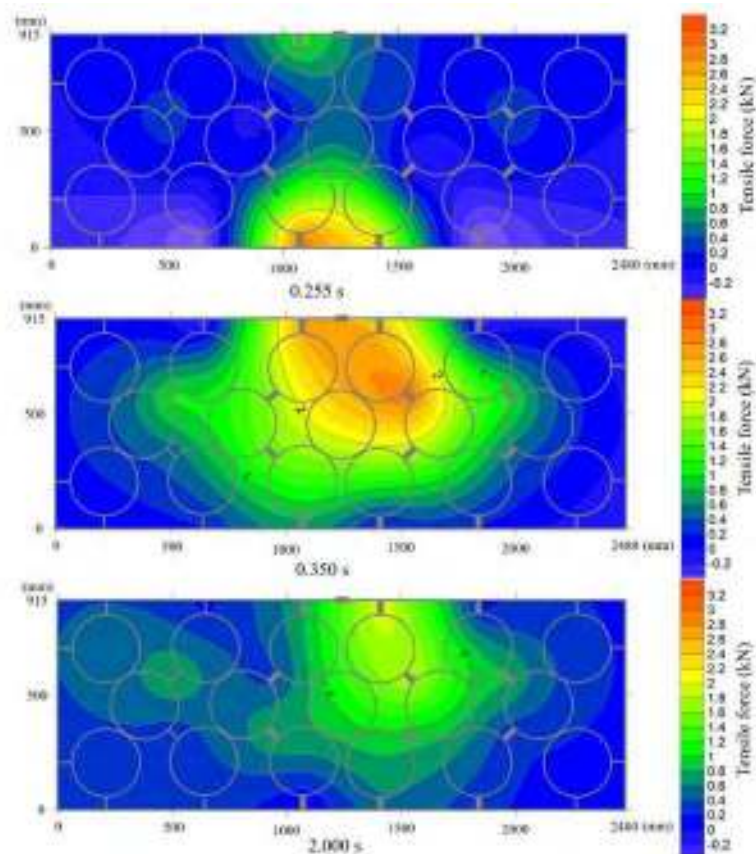


Figure 4: Distributions of the tensile forces on the flexible barrier at typical time points

From the impact force histories of Transducers 1 to 12 in the debris flow impact test in Appendix A, the peak loadings of all mini-tension link transducers are plotted in Fig.5. From the figure, it can be observed that the maximum impact pressure on the central area is much larger than the value on the side area instead of uniformly distributing on the flexible barrier, which fits well with the conclusion drawn by Wendeler et al. (2018) from back-calculation using the data of field-scale debris flow impact tests.

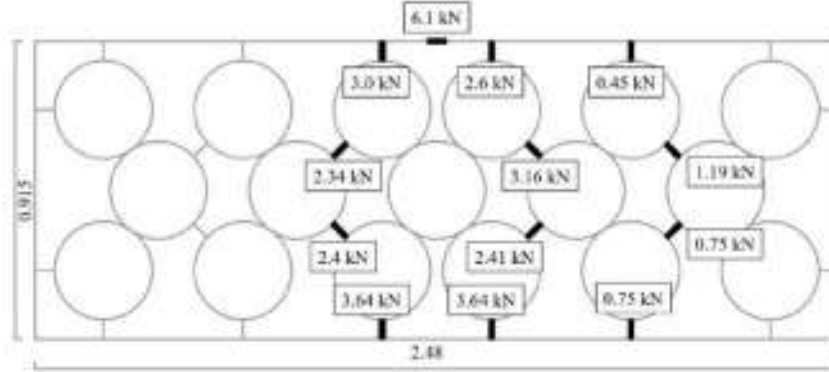


Figure 5: Peak tensile forces of all mini-tension link transducers on the flexible barrier in the test

4 A NEW SIMPLE METHOD FOR CALCULATING DYNAMIC IMPACT FORCE

Based on the observation of the test in this study, a certain percentage of debris material passed through the flexible barrier and reduced the impact force on the flexible barrier. Considering the influence of partial debris passing-through on the impact force, a new simple method is proposed to calculate the dynamic impact force based on the hydro-dynamic method raised by Armanini (1997). In this method, a debris flow is simplified into a two-phase flow to represent different impact characteristics of the trapped particles (debris phase) and the passing slurry (slurry phase). It is assumed that both components are uniformly distributed in the debris flow with the same velocity. The impulse on the flexible barrier is calculated from the momentum changes of the debris phase (Δp_d) and the slurry phase (Δp_s):

$$F_{\text{impact}} t_{\text{impact}} = \Delta p_d + \Delta p_s \quad (2)$$

where (F_{impact}) is the dynamic impact force of the debris flow in the impact period (t_{impact}).

Two parameters, named Velocity Loss Ratio (VLR) and Debris Retaining Rate (ϕ_d), are correspondingly introduced to quantify the velocity loss of the passing slurry and the percentage of the retained debris, respectively. The parameters are expressed as follows:

$$\phi_d = \frac{V_d}{V_{\text{mixture}}} \quad (3)$$

$$\text{VLR} = \delta = \frac{V_0 - V_{1,s}}{V_0} \quad (4)$$

where V_d and V_{mixture} are the volumes of the trapped debris material and the debris flow mixture in the impact period, v_0 is the impact velocity of the debris flow, and $v_{1,s}$ is the residual velocity of the passing slurry.

By assuming that the velocity of the debris flow is a constant (v_0), and the impact area is a rectangle with the area of the channel width w multiplying the debris depth d , the simple method for impact force calculation can be written as:

$$F_{\text{impact}} = v_0^2 [\phi_d \rho_d + (1 - \phi_d) \delta \rho_s] h w \quad (5)$$

Utilizing this simple method, the dynamic impact force from a debris flow on a flexible barrier can be calculated with basic parameters: the density of the debris phase (ρ_d) and the slurry phase (ρ_s), the impact velocity (v_0) and the flow depth (h) of the debris flow, the width of the flowing valley (w), and two dimensionless coefficients with specific physical meanings: the velocity loss ratio (δ) and the debris retaining rate (ϕ_d). Detailed deduction of the simple method is presented in Tan et al. (2019).

The passing-through phenomenon has been frequently observed in previous field tests and real cases (DeNatale et al. 1999; Geobruagg AG 2012). However, few data in the literature have measured the velocity of the passing debris flow and the retaining rate of the debris in the test. Therefore, we carefully measured and analysed the data in the performed physical modelling experiment to verify the proposed new simple method. The maximum dynamic impact force on the flexible barrier was calculated using Eq.1. The depth of the approaching debris front and the velocities of the debris flow before and after the impact were measured from continuous photographs taken by the side-view high-speed camera. All relevant data are listed in Table 1.

By integrating the parameters of the test into the new simple method, the calculated impact force using the new simple method is 10.94 kN, which fits well with the measured impact force on the flexible barrier of 10.35 kN (see Table 1). Therefore, the new simple method is validated and can be used to calculate the dynamic impact force of a debris flow on a flexible barrier. However, more experiments are needed to further investigate the influencing factors of the simple method before it can be utilized in the design of debris flow-resisting flexible barriers.

Table : Parameters and the measured values of the debris flow test

Parameters	Debris impact test
Density of debris phase ρ_d (kg/m ³)	2075
Density of slurry phase ρ_s (kg/m ³)	1655
Bulk density of the debris flow ρ_{bulk} (kg/m ³)	1739
Water content (%)	46.5
Total volume of debris flow mixture $V_{mixture}$ (m ³)	2.05
Retaining volume of debris materials V_d (m ³)	0.64
Retaining rate ϕ_d	0.31
Velocity of debris flow before impact v_0 (m/s)	5.8
Velocity of debris flow after impact $v_{1,s}$ (m/s)	2.6
$Fr = \frac{v_0}{\sqrt{gh}}$	4.49
Velocity Loss Ratio δ (%)	55.2
Depth of debris flow h (m)	0.17
Width of the flume w (m)	1.5
Included angle θ (°)	127
Measured impact force $F_{measured}$ (kN)	10.35

5 SUMMARY

A debris flow impact test was performed to study the interaction of a debris flow with a flexible barrier. Thirteen mini-tension link transducers were installed on the flexible barrier to measure the impact force distribution on the flexible ring net in the impact process. Two high-speed cameras were utilized to capture the interaction of the debris flow with the flexible barrier. The major findings from the study in this paper are presented as below:

It is found from the physical modelling test that the impacting time of a debris flow on a flexible barrier was less than 1 s. It was observed from the photographs by the high-speed cameras that a large percentage of small particles and slurry passed through the flexible barrier at the initial stage. The

impact force distribution on the flexible barrier during the impact process was presented by visualized contours in this paper. From those contours, the impact loading on the central area of the flexible barrier was much larger than the loadings on two sides. During the interaction, the debris flow firstly impacted on the central bottom area of the flexible barrier, then the major impact area rose up with the deposition of the debris material. It should be noticed that the scale of the debris flow in this study (2 m^3) is still not as large as the real debris flows in Hong Kong with the volumes normally larger than 500 m^3 (King 2013).

Moreover, based on the passing-through phenomenon observed in the impact process, a new simple method is proposed using the impulse-momentum theorem. In this method, permeability is emphasized as a key parameter to calculate the impact force more accurately. Two parameters with specific physical meanings, named Velocity Loss Ratio (δ) and Debris Retaining Rate (ϕ_d), are introduced in this simple method. This method has been validated by comparing with the measured impact force in the physical modelling test. After a systematic calibration of those coefficients, this method can be utilized as a more accurate method than the hydro-dynamic method recommended in DN 1/2012 and the energy capacity utilized in TGN 37 in design analysis of a debris flow-resisting flexible barrier (Kwan and Cheung 2012; GEO 2014).

ACKNOWLEDGEMENTS

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APPENDIX A

From Fig.1(b), Transducers 1 to 8 were used to measure the impact loading on the central area of the flexible barrier. Transducers 9 to 12 were installed on the right side to measure the impact loading on the side area of the flexible ring net. Transducer 13 was installed on the cross-tension cable of the flexible ring net to measure the tensile force on it during the impact. The cross-tension cable was used to stretch the ring net and transfer the impact loading to the supporting structures. The force histories of all transducers are plotted in Fig.A1. Dynamic impact forces are observed in the signals of Transducers 1 to 8 with the peak loadings ranging from 2.4 kN to 3.64 kN. As a contrast, the tensile forces on Transducers 9 to 12 are much smaller than the forces on the transducers installed in the central area. For the force history of Transducer 13, which represents the tensile force on the cross-tension cable, two dynamic impacts with similar peak values around 6 kN are observed. The static loading after impact is also relatively high with the value of 5.5 kN.

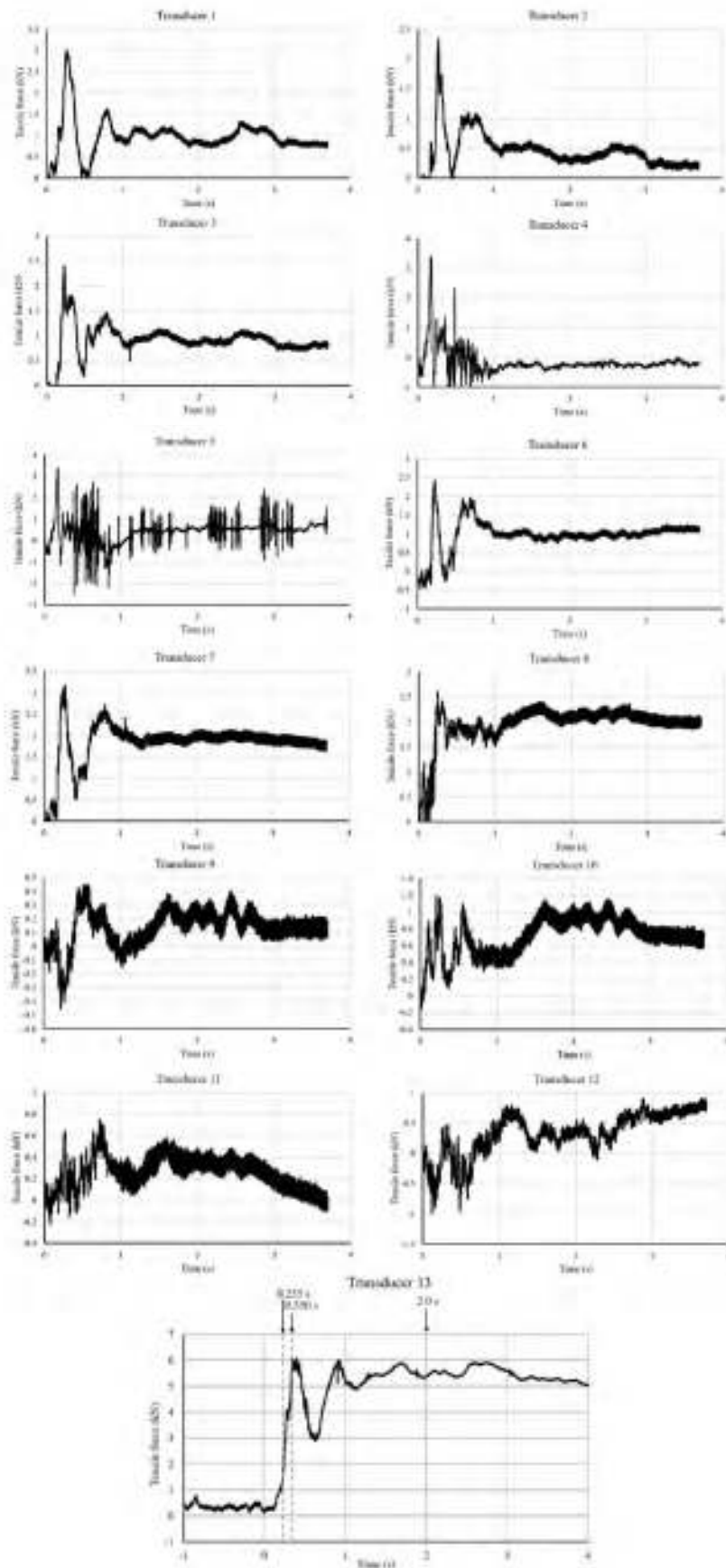


Figure A1: Tensile force histories of thirteen mini-tension link transducers in the debris flow impact test

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ISO 19650 – A International Approach to the Journey of Industry Integration, Digitalization, and Innovation

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ABSTRACT

It is forecasted that by 2030, the global construction market will increase by 8 trillion US dollars compared with 2018, and the output of China's construction industry will increase by 2.1 trillion US dollars. In a time when China's construction industry is facing tremendous opportunities and great changes regarding digitalization and other digital technological changes, the development and adoption of the UK BIM Level 2 (PAS1192) has been a critical step on the journey to build a digital world for the construction industry overall. Level 2 was a critical starting point for the UK sector development strategy to drive collaboration, integration, and digitalization. This UK national standard has significant meaning at high-level information management through-life for a built asset. PAS1192 has now been in use for a number of years and has matured into ISO 19650. This is an international BIM standard that contains all the same principles and essential requirements of BIM Level 2 and was published in the first quarter of 2019. This paper records the process of the first implementation of ISO 19650 into a public building project in Hong Kong. Through this case study, it demonstrates that the use of an international standard is found to be essential not just to cut costs and to improve the productivity of designers, engineers, and operators, more importantly, to drive the readiness journey to BIM Level 3.

1 INTRODUCTION

1.1 Global AECO industry

With the rapid development of global economics, the public has more and more demand for both residential and public buildings and assets. However, the low productivity of the construction industry has been emphasized for a long time. The productivity of the construction industry is much lower than the total economy since 1995 (Economist 2017). Many construction projects are over budget and behind

schedule. Through the advancement of technology, many sectors are entering into a digital world. Several technical studies demonstrated that the Architecture, Engineering, Construction, and Operations (AECO) industry has the lowest sector digitalization level. A large number of AECO enterprises did not realize the benefit and urgency of digitalization and only a handful of enterprises were planning to move to a digital world (McKinsey Global Institute, 2017). However, the analysis from McKinsey clearly indicated that AECO industry is “ripe for disruption” and two exhibits were cited:

- For large construction projects, 20% of them are over schedule and 80% are over budget. Cost and schedule overrun has almost become a normal condition.
- Compared with other sectors for example in Germany and the UK wider economy, labour productivity of the AECO industry is significantly lagging.

Project delay and low productivity are the main reasons for relatively low financial returns of the AECO industry, while most researchers indicated that the underlying causes of project delay and low productivity are attributed to low level digitalized process and less innovated technology in AECO projects. The root cause indeed is the lack of high-value information in comparison to the engineering sector (Tang et al. 2010, Zhao et al. 2007). The approach advocated (Bew, Underwood) was later echoed in the UK Construction Strategy 2011, calling for a “ramping” up of capability through the definition of the Levels approach.

1.2 Research objectives

This paper records the process of the first pilot implementation of ISO 19650 into a public building project in Hong Kong. Through this case study, it is demonstrated that the use of an international standards language is found to be essential not just to cut costs and to improve the productivity of designers, engineers, and operators, more importantly, to enable the journey of BIM Level 3 (Bew and Underwood 2010) through life-cycle integration of information, digitalization, and innovation. Section 2 sets the scene introducing some of the latest development of ISO 19650 and some related local HK BIM standards/specifications. Section 2 looks at how the latest published ISO 19650 can be implemented in a case study public project in Hong Kong. Its sub-section also introduces the project background of the case study public project in Hong Kong. Section 3 is the major part of the work analyzing how ISO 19650 drives the data integration and the creation of a Project Information Model (PIM) along the life cycle of the case study project using many BIM international languages. The final section closes out the paper with a number of gaps for the development of a local BIM Annex for HK AECO industry regarding the implementation of ISO 19650. It also shows how the journey to BIM Level 3 is based on the progressive development, learning, implementation and growth of capacity to meet market demand.

2 ISO 19650 IMPLEMENTATION IN THE HK AECO INDUSTRY

In late 2018, the British standard BS 1192:2007+A2:2016 and PAS1192-2:2013, which are included in BIM Level 2 strategy in the UK, were promoted to the new ISO standards BS EN ISO 19650-1 and BS EN ISO 19650-2 (BSI Group 2018). It is marked as a significant step forward for the BIM-based information management requirements to be an internationally agreed set of concepts and principles (BSI 2019). The updates in the specific terminology and language are the fundamental changes for the UK documents’ transition to the international standards.

ISO 19650-1/2 cover the concepts and principles and delivery phase that need to be implemented in project and asset management parts throughout the project lifecycle (ISO 2019a; ISO 2019b). ISO 19650-2 includes the ‘UK National Annex’, providing additional recommendations based on the UK market to better align with international standard (BSI 2019). The UK annex is not required in the global environment and other nations are encouraged to explore and document their local guidance and suggestions, to support the information management in the building lifecycle to improve the industrial standardization and be applied in local projects (ISO 2019b).

This raises the questions for the HK AECO industry; does HK need to compile the local ‘Annex’ for ISO BIM and how the ‘Annex’ should be developed based on existing BIM standard and standards in HK? HK Government organizations, such as CIC (Construction Industry Council), HKHA (Hong Kong Housing Authority) and EMSD (Electrical Mechanical Services Department), have issued several BIM

standards about project or asset management in recent years. Most of them are technical and practical standards, which play different roles with the market and the ISO BIM standards landscape. Apart from standards mapping, to conduct an appreciated Annex for the HK market, the standard needs testing in the local market to establish how well it fits with local customs, regulations, best practice and contract conditions. This paper does not only explore the process of ISO BIM implementation in the local market, it also provides the foundation and justified research information of the Annex formulation in the HK AECO industry.

2.1 Case Study Project Background

To further explore the impact of ISO 19650 to HK AECO industry, this study takes a new public project in HK as a research case. It is a three-story building with around 400m² built on a complex and sloping site. The primary functions of this building are to preserve and exhibit the historical relics for the public and also provide a comfortable place for people to study, read and relax. The research team has achieved a close connection with the project and client team and visited the site to review use of ISO19650, and the BIM federated model, including architecture, structure, mechanical, electrical and plumbing (MEP) and interior decoration systems during the design phase, which is the PIM (Project Information Model). The work of clash detection, 4D scheduling, and 5D costing has been undertaken to assist the decision-making during project delivery phases, see section 3.2. Also, to assist in the handover process, the AIM (Asset Information Model) is updating simultaneously with engineering changes, which will be utilized in the Facility Management (FM) phase.

3 THE IMPACT OF ISO 19650 IN THE HK AECO INDUSTRY

It has been observed that the application of PAS1192-2/3 has increased in recent years and there has been good results for the BIM level 2 implementation in the UK market (NBS 2018). The new international BIM standards have been developed from PAS1192-2/3. Based on the local market conditions in the HK AECO industry, the impact of ISO 19650 is explored from the perspectives of migrating through the lifecycle integration, digital twin and cross-industry innovation phase as described in Figure 1. To promote the performance of the AECO industry, the primary step is to integrate information delivery along the whole of the asset lifecycle. Clear and precise information requirement is essential to lead the downstream project activities and to establish the accurate and trustable digital twin. A Digital Twin is the virtual presence of the physical asset, which can support the decision-making during building design, construction and operation stages. In this paper, we focus on the cross-industry application standards to move us from BIM towards the digital twin originally described by Bew and Underwood in Level 3. In this section, the information management workflow in ISO19650 is identified firstly through the perspectives of lifecycle integration and digital twin establishment, and then the corresponding documents in current HK AECO industry are mapped to show the potential gap of establishing the local BIM Annex based on the practical challenges from a contractor and client perspective.

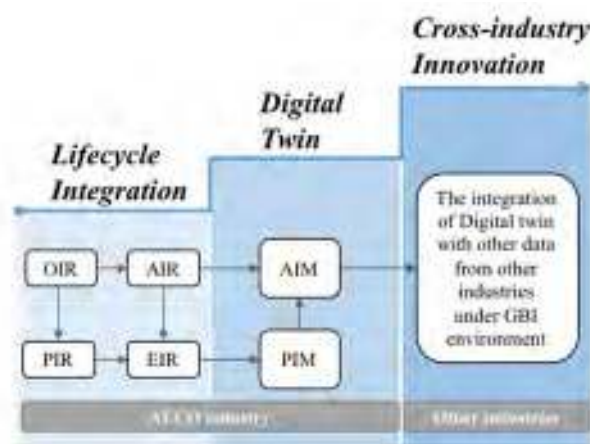


Figure 1: The impact of ISO19650 on HK AECO industry

3.1 Lifecycle Integration

AECO is a project-based industry with a large number of diverse professions, specialists and suppliers and every construction project is unique to some extent (Murdoch et al. 2017). For this reason, to efficiently manage a project, it requires specific requirements from clients, sponsors and other stakeholders to guide the whole lifecycle information delivery process. The requirements must be considered from the perspectives of objectives in the building in-use phase and be delivered from the project beginning stage. Through properly procuring the requirements, the appointed parties can be integrated systematically via defined information exchange activities. The role of appointing party is important, who should take the responsibility to confirm the specific requirement, such as project positioning, ultimate goal, and resources organization (Bosch et al. 2015). However, if there is no explicit requirement from the appointing party, the information fragmentation caused will create an inefficient, tedious and expensive information acquisition and management activity. In other words, clear requirements from the appointing party can provide strong support for the effective completion of the building and its future lifecycle management. What information should be collected at which stage, in which manner and who should take the responsibility of it should be programmed logically (Parsanezhad 2014), otherwise, the value of BIM will not be realised (Grilo & Jardim-Goncalves 2010).

At the project commencement, the project appointing party should ensure high awareness of the objectives of building operational stage (Cavka et al. 2017). Only in this way, the downstream teams can identify the level of need of the information (Pärn et al. 2017). It has been identified that unclear information requirement is one of the most significant reasons that will lead to the disordered lifecycle information routine throughout the whole building lifecycle. ISO 19650-1 (ISO 2019a) emphasizes the significance of the information required. It defines the four types of 'requirements' and describes the relationship between them, as shown in Figure 2.

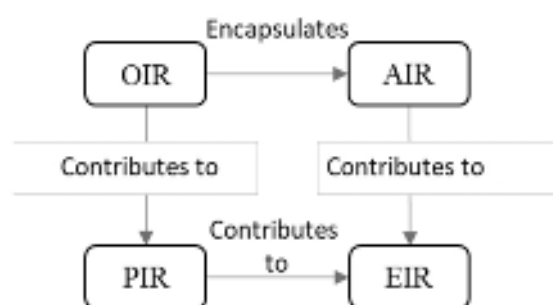


Figure 2: Information requirement circle in building lifecycle (ISO 2019a)

OIR and AIR: At the earliest stage, OIR (Organizational Information Requirements) should be identified documenting the information needed to answer or key operational or governance questions or controls relating to the services the asset or network in question being provided by appointing party or his down stream operations team requirements.

The AIR (Asset Information Requirements) is more specifically regarding the managerial, commercial and technical aspects of a single assets information requirements. The information required in AIR should answer the asset-related part in OIR. Table 1 shows the mapping condition between the requirements in ISO19650 and project service stages defined in RIBA PoW (Plan of Work) (RIBA 2013). In the table, '○' means that the specific information requirement should be generated thinking from the needs in the corresponding phases and '●' indicates that the requirement should lead the information delivery process in the above stages. Referring to the definition in the standard, both OIR and AIR should be generated considering the objectives at post-construction stages. The differences between them are that OIR much more focuses on the business destination of the whole project, such as the finance accounts aspect, and AIR pays more attention to management on the actual in scope asset. However, both the OIR and AIR should be settled and delivered before the design phase commences to lead the following activities throughout the building lifecycle. Once created the documents or data sets can be updated through formal change control.

PIR and EIR: The term PIR (Project Information Requirements) are the specific information sets

that need to be procured in the contract during the deliver and handover process to manage the deliver process and satisfy the AIR and OIR. Sometimes the use of PLQ (Plant Language Questions) based on PAS1192-2:2013 (BSI 2013) can help synthesis the derivation of these data sets. The PLQ/PIR describe the requirements for all information exchange points and transfer responsibility for servicing these exchanges to the supply chain through the contract. The new definition of EIR in ISO 19650 is 'Exchange Information Requirements'. Comparing to the previous 'Employer's Information Requirements' in PAS1192-2, the new explanation emphasizes more about the information exchange activities through all stages, mainly before the occupation phase, see Table 1. As shown in Figure 2, EIR is contributed by PIR, which should specify the detailed information needed to answer in PIR. Besides, PIR and EIR are contributed by OIR and AIR to deliver the requirements further. The language used in the ISO is not helpful as it introduces yet more terms to an already complex process. These terms require aligning with the local HK contract terms, which mostly align with UK and old PAS terms.

To conclude the information requirement model, it can be identified that OIR is the overall requirement of a project and AIR, PIR and EIR need to fulfill the need of the OIR. The EIR must deliver the detailed requirements in every requirement, in order to realize the objectives from the primary appointing party (OIR). Figure 2 and Table 1 show the information requirements circle for each built project. It is also addressed in ISO 19650 that the language of 'information requirements' does not just indicate the information to be delivered, the information delivery standard, resources, appointing/appointed parties and milestones are also key aspects to be considered. The broader meaning of the 'information requirements' realizes the project lifecycle integration through managing the quality of information and information exchange details, so that the high-level business objectives can be achieved.

Table 1: Mapping between information requirements in ISO 19650 with project service stages defined in RIBA PoW

Service stages BIM Language	Strategic definition	Preparation and brief	Concept design	Developed design	Technical design	Construction	Handover and close out	In use
OIR	•	•						○
AIR	•	•					○	○
PIR	•/○	•/○	•	•	•	•	•	•
EIR	•/○	•/○	•	•	•	•	•	•

The standard of project model, workflow and supplier capability assessment should be included in the Bim Execution Plan (BEP), this document, which provides a practical description of the BIM adoption process at the project preparation stage. The EIR should be considered based on OIR (Organizational Information Requirements) and AIR (Asset Information Requirements). Both of these documents describe the requirements of asset management strategy in the operational period. The appointing party is at the core of the temporary project alliance; the contract will define the relationships that can encourage the whole group to hold a common understanding of the project and enable collaborative working. A well-structured information requirement system is the precondition to manage the fragmentation of information during the whole building lifecycle.

To evaluate the value, opportunities and challenges for ISO implementation on projects to promote project integration, the research team investigated a case study project, to develop the mapping table of information requirements between ISO 19650 and the requirement through the lifecycle of the research case, see Figure 3.

By way of background, the project selected is a public project for both relic exhibition and storage. The objective of the procuring party is to protect and preserve the precious cultural relics in high quality. To respond to the requirements, the AIR and PIR describe the requirements profoundly considering the aspects of asset and project management. The AIR describes all functional aspects including the security requirements at the operational stage, for example, the deployment of fire-fighting equipment, CCTV and other alarm facilities. To better preserve the relics, the indoor environment also needs to be carefully regulated. The design of the environmental sensor structure is another aspect described in the

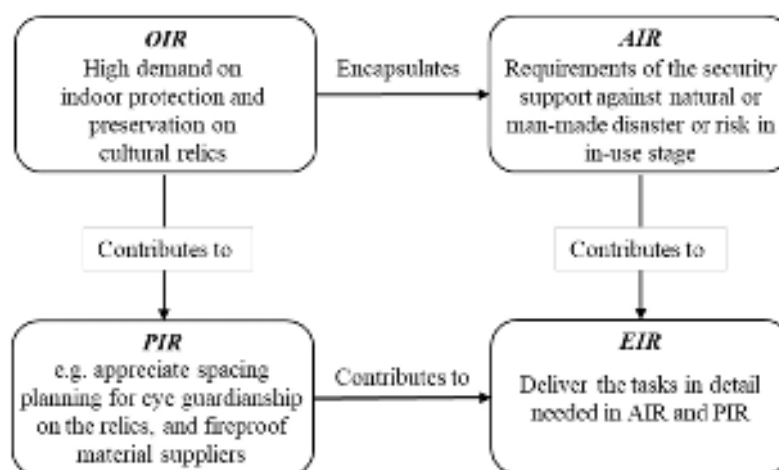


Figure 3: Information requirement mapping between ISO 19650 and the case study in HK

AIR. To make sure the relics be safely exhibited among the public, the spacing planning and the specific fireproof built material is be considered in PIR. As to EIR, it includes the information defined by the AIR and PIR, leading the realization of OIR eventually.

In this paper, the research team investigates the project background and proposes a structural mapping with the information requirements defined by ISO 19650. With a clear information requirement route, the organizational objectives can understood in higher quality comparing to traditional methods which provide less awareness of the project OIR among project parties. Because of the disordered information management, people traditionally need to do repeat work on collecting useful building information in the operational stage, wasting a tremendous amount of workforce and material resources (Teicholz 2013). The role of appointing a party to issue the OIR plays the most crucial role in this process, which is the most significant challenge. To realize the value of ISO 19650 to facilitating the integration level among the distinct parties, the owner should issue clear objectives to guide the follow-up information exchange activities.

3.2 Establishment of Digital Twin

Early in the 1960s and 70s, NASA has used the ‘mirrored systems’, which is the early form of digital twin (Hamilton 2018). From aerospace to the AECO industry, people construct buildings in the physical world and digital twin based on BIM technology is the digital presence of the corresponding constructed buildings in the virtual world. It is the realistic reflection of the process, product or service within the project in the real world (Hamilton 2018). The concepts of PIM and AIM are emphasized in ISO 19650-1 and both of them can be regarded as early forms of a digital twin throughout the building lifecycle (ISO 2019a). As shown in Figure 4, PIM is specified from EIR and AIM should meet the requirements in AIR. At the project handover stage, the PIM should contribute to AIM with the specific data being exchanged across the asset management and operations activities. This section introduces the functions of PIM and AIM during the project delivery phases, also with their presentation in the target case. The digital twin should keep synchronous adjustment according to construction/ maintenance changes in the actual physical assets. ISO 19650 identifies the CDE (Common Data Environment) as a logical method to provide a collaborative working environment delivering the right information for project parties.

PIM: PIM should support the activities during design and construction phase and to enable the handover phase, the PIM should be archived for the further auditing (ISO 2019a). In the researched project, the team has established the PIM for the project and to explore more potential of PIM, the function of clash detection (Plate 1), 4D scheduling (Plate 2) and 5D costing (Plate 3) have also been tested.

In the case study, the changes in early design and construction stages have been simulated in PIM to avoid clashes as detected. Through applying 4D and 5D, although there was an extension of completion time of three months due to inclement weather and other design changes, the cost has been controlled well within contractor’s estimation and project budget.

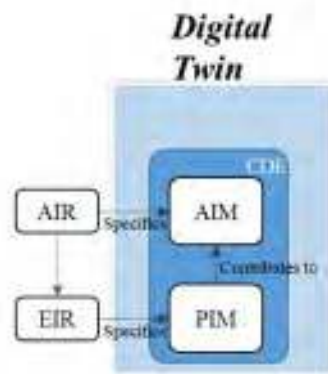


Figure 4: Defining a Digital twin

CDE: The digital twin should be built synchronously with the real world design, construction and building operational processes to make sure the accuracy of the digital asset information (Cavka et al. 2017). Due to engineering changes, a large number of information sources can easily cause a fragmentation and poor coordination problem. The concept of CDE – Common Data Environment is put forward in standard ‘BS 1192:2007 Collaborative production of architectural engineering and construction information – Code of practice’ (BSI 2007), which has been transferred to ISO19650-1 (ISO 2019a). The CDE is regarded as a repository providing agreed source of information to process all project documents, non-geometrical data, and geometrical models. The ‘transitions’ in CDE provides information audit mechanism ensures the information accuracy and enables all parties to work in a collaborative environment.

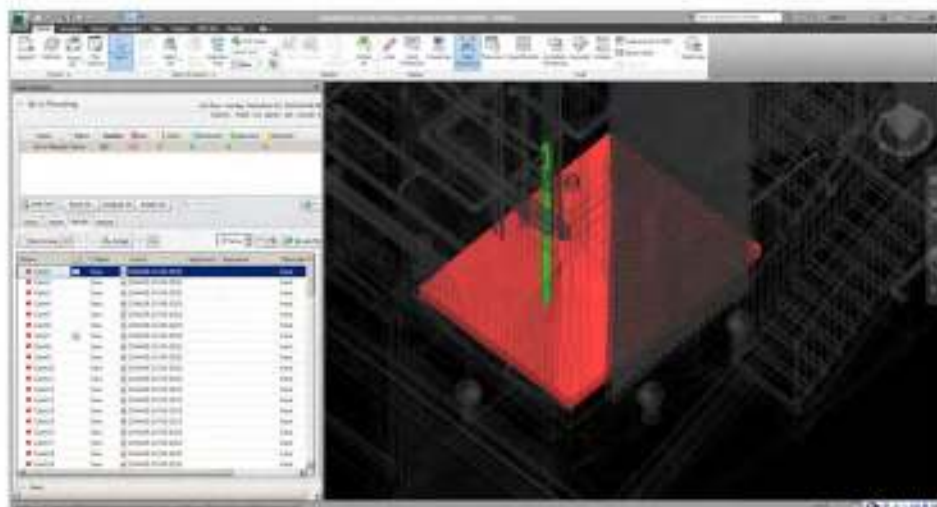


Plate 1: PIM for clash detection through Revit



Plate 2: PIM for 4D scheduling through Navisworks and Microsoft Project



Plate 3: PIM for 5D costing through Vico

3.3 National BIM Annex in HK AECO industry

It is suggested in ISO 19650-2 that the national standard bodies should compile and document the related information management standards into the national Annex, which are used to provide specific guidance and advice on how to adopt the ISO BIM standard in varied projects (ISO 2019b). In this paper, the local BIM related documents and standards, which are bid requirement for the main contractor PS.G 17 and CIC BIM standard phase one by CICHK (CICHK 2015), are mainly considered comparing with the UK National Annex in ISO 19650-2. The information requirement and management methods before project handover phase (the procedure from EIR to PIM) are the critical mapping points to indicate the gaps of BIM Annex in the HK market. The interview with the main contractor of the case project is the principal source to understand the BIM adoption in the HK market.

In ISO 19650-2, requirements are demonstrated in different information delivery phases. While in CIC BIM standard, the main requirements are divided into the BEP, model, LOD and data organization, and the requirements in PS.G 17 are put forward mainly according to the information deliverables. Comparing to the ISO 19650-2, the two specifications from the HK market emphasize much more on the perspectives of technical details and deliverables, which is a bottom-up approach. The team mapped the requirements targets groups among the three standards and documents, see Table 2, and it can find that there are fewer information requirements to owners, sub-appointed parties and task groups in CIC BIM standard and PS.G 17. Further, the team explores in details on the information requirements and management among them. Three key gaps are concluded after the content mapping:

1. Information audit
2. Requirements to owners
3. Requirements to sub-appointed parties/ task teams.

Information audit: The requirements of information audit in HK standards are mainly focused on the technical contents, such as model compliance. However, the concept of 'decision point' is emphasized in ISO 19650-2, which means there are clear requirements on the time points of the information audit with clearly defined project delivery phases. Besides, the clear information containers in CDE defined in ISO 19650-2 is also an important method to improve the information quality. Through the interview with the main contractor of the case study project, it is mentioned that all works and submission should be uploaded to a shared drive; however, there is no standard and clear procedure to check the BIM deliverables and the information quality cannot be guaranteed efficiently. The clear defined 'decision points' and the design requirements of CDE to make sure that the qualified information delivered is essential in the information management workflow.

Table 2: The requirements target groups in the three standards and

Standard & Document	Requirements to	Appointing party	Lead appointed party	Sub-appointed party	Task team
Information delivery phase of ISO 19650-2					
1. Assessment and need		✓			
2. Invitation to tender		✓			
3. Tender response			✓		
4. Appointment			✓	✓	
5. Mobilization			✓	✓	
6. Collaborative production of information					✓
7. Information model delivery			✓	✓	
8. Project Close-out		✓			
CIC BIM standard phase one by CICHK					
Client requirement specification		✓			
Model methodology & LOD & Component presentation style & Data organisation			✓		
Particular specifications PS.G 17			✓		

Requirements to owners & sub-appointed parties/task teams: Through the HK case study investigation, it is found that the primary target group of BIM information requirement is the main contractor. The main contractor needs to submit management and coordination plans stating the relationship with subcontractors on BIM tasks. However, there is no standard content about the requirements to the sub-contractors or other downstream parties. More requirements should be raised to all involved parties to realize the collaborative working environment among the industry. As to the appointing parties, except for the requirements that should be put forward to the downstream parties, the self-review should also be considered, for example, the assessment of the project resources and the clear criteria to BIM deliverables, which are emphasized in ISO 19650-2. Besides, the concept of 'mobilization plan' is raised in ISO 19650-2 to distribute the requirements to the subdivided parties, such as sub-appointed parties and single task groups. This is the potential gap to be filled on only the main contractor can receive the BIM requirements in HK standards and documents. Only the BIM standards and documents can involve all project parties, and the BIM application can reach the maximum value during the information delivery process.

Table 3: Gap mapping among the specific standards

Standards	ISO 19650-2	CIC BIM standard	PS.G 17
Gaps			
1. Information audit	Clear decision point	Technical requirement of the compliance check	Technical checklist for deliverables
2. Requirements to owners	Allocation of resources and criteria of deliverables	Lack of self-review	N.A
3. Requirements to sub-appointed parties/task groups	Mobilization plan for lead and sub-appointed party	N.A	N.A

In the case study project, the ISO 19650 has been implemented and demonstrated in this paper. The use of BIM has significantly improved the planning at the design stage and programme and cost controlling at the construction stage. However, difficulties have been encountered at the data transfer between parties, which reflect the three gaps mentioned in this section. Besides, the real strategy of information requirements and management throughout the delivery phase in ISO 19650 is the collaborative working among all the involved project parties. So that the information updates on cost and time in the model of

construction stage could be reflected timely, this is the essential and valuable knowledge for most of the local practitioners before they understood the benefits of implementing BIM on their projects. The BIM Annex in HK should be compiled to fill the gap to realize the value of BIM, at the same time, to meet the international BIM requirements.

4 FUTURE STUDY

Future study should focus on the case study project from lifecycle integration to digital twin establishment as defined in the Level 3 definitions. Further the specifics of the HK trading and contract specifics including regulation and local language should be further considered on both buildings and infrastructure projects to establish the basis for a full HK ISO Annex.

5 CONCLUSIONS

This paper records the process of the first pilot implementation of ISO 19650 into a public building project in Hong Kong. Through this case study, it demonstrates that the use of an international standard is found to be essential not just to cut costs and to improve the productivity of designers, engineers, and operators, more importantly, to drive the journey of BIM development through life-cycle integration of information, digitalization, and innovation. The major contribution of the work analyzing how ISO 19650 drives the data integration along the life cycle of the case study project using different BIM international languages. It sets a solid foundation for HK to consider to need to develop 'Annex' for ISO BIM version for HK. The paper also puts forward the gaps to establish an Annex for HK BIM through mapping the information requirement and management strategy in ISO 19650-2 against different local standards.

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Integrated Application of Shoring Modularization, Digitalization and Monitoring Sensors in Deep Excavation Project

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ABSTRACT

With increasing demand on the time and cost effectiveness as well as environmental friendliness in construction projects, standardization of structural members facilitating quick handling and reuse has become more and more popular. Together with the advancement of BIM and other digitalization technologies, more sophisticated design and planning is available for promoting modularization. The use of sensors to monitor the forces of members also helps to ensure safety and provide confidence with adequate verification of design. This paper presents an example of the integrated application of shoring modularization, digitalization and monitoring sensors to facilitate the ELS design and construction.

1 INTRODUCTION

Modularization and standardization in construction is not a new idea in the industry. The concerns may have been on the difficulty of starting up and the establishment of sustainable business line for maximizing benefits. With advancing digitalization technologies, the design and planning of modularization is becoming easier and more cost effective, therefore promoting the pursuance of modularization together with the demand in environmental friendliness. The example of the integrated application of shoring modularization, digitalization and monitoring sensors is hoped to showcase the potential benefits and promote the integrated delivery as a norm.

In the design of the modular members, the full process and construction cycles have to be taken into consideration, from fabrication, transport, installation, compatibility with other structures, effective utilization, to removal and reuse in other projects. Highlighted features in the subject application includes:

- Strategic selection of member sizes for ease of identification on site
- Bolt connection design with consideration for ease installation and inspection
- Integrated edge protection system compatible with strut orientation and section sizes
- Pre-fabricated steel spacer to eliminates site welding

To allow the design to be effectively implemented in construction, the aid of Building Information Modelling (BIM) and 3D modelling plays an important role in planning and design modification. Here, 3D modelling with a model-based construction software is used for virtual assembly constructability study and fabrication. The BIM incorporating the temporary works and permanent structures is used to identify and solve the clashes between the shoring and permanent structures. With numerous modular components on site, QR codes help with the resource management.

Wireless strain gauge sensors are used to monitor the strut forces, providing data of actual structural utilization and acting as safety assurance. The modular members at critical locations are closely monitored and the designed and actual utilization at each stage of construction can be compared.

Overall site photogrammetry models are also generated regularly with drone taken aerial photos as part of overall monitoring records. The benefits, obstacles and prospects of the integrated application of modularization with technology will be further discussed in the paper.

2 SHORING MODULARIZATION

2.1 Design for Manufacture and Assembly (DfMA)

Design for Manufacturing and Assembly (DfMA) applies the principles of utilizing assembly line manufacturing in construction with whole-cycle planning for significantly reduced programmes, enhanced quality and improved safety on site. In DfMA, integrated modules and components of a structure are fabricated off-site in a factory, by highly efficient machines or skilled labours under stable and suitable environment, then transported to the construction sites for rapid and precise installation. The aim is to establish an efficient production line, reducing waste and risks through minimizing raw materials handling and labours on site, as well as lowering cost and carbon emissions by facilitating reuse and recycle of components and materials within the circular economy.

The idea may be better understood and implemented for Modular Integrated Construction (MiC) in building works in Hong Kong. Temporary works such as shoring for deep excavation, however, could have even higher potential to develop with the concept, provided the flexibility against design change, fewer concerns on approval compared with permanent works, and the need of considering reuse and recycle with short life span in individual projects. An example of application of DfMA in shoring modularization for a deep excavation project is shown in Plate 1 and the following discussions will be based on this example.



Plate 1: Application of DfMA in shoring modularization

2.2 Standardization

Standardization is the key for successful DfMA application in shoring. There are proprietary products in the market providing numerous choices of member types and sizes. The strategy here, on the other hand, is to develop an own system with strategic selection of member sizes and component lengths. The considerations include suiting the commonly adopted ELS design approaches, fitting the manufacturing process, being convenient for logistics, with ease of identification and installation on site, as well as considering the typical ranges of members applicable to typical Hong Kong projects. This not only facilitates the standardization in a single project during construction, but also allows flexibility for the standard to be applied in most other similar projects through the whole production cycle.

Sizes are designed in 2 major types – large and small with restricted variation in member sub-types. The typical design works and manufacturing can then be kept simple, while the key issue is the ease of identification and storage on site. It is less likely to mix up different module types and could avoid additional lifting to pick the correct modules on site. Typical lengths of modules are standardized for the ease of management, with the consideration of easy logistics while limiting the total number of connections required. The connection design aims at simple, fast and reversible for high efficiency during installation and dismantling. Bolt connection design is selected fitting the purpose (Plate 2), also simplifying the required inspections. On-site welding and flame cutting works can be minimized.

In addition to the major member components, the design of appendage to the shoring members are also considered to form an integrated system. Multi-functional lifting eyes are incorporated during fabrication of members. Pre-fabricated steel spacer to the waling members is designed to eliminate site welding. Integrated edge protection system compatible with strut orientation and section sizes is also developed (Plate 2), allowing safe access making use of the members.



Plate 2: Bolt Connection between components and integrated edge protection compatible with strut orientation

2.3 Safety

In increasing productivity, modularization does not compromise safety, in contrast considering it as a major objective. The changes brought in quantity and quality of works lead to improved safety. Fabrication in factory is maximized and installation works on site are minimized. Factory procedures are under stabler environment while construction sites are dynamic, the quantity shift of works towards a more suitable environment allows risks to be better controlled. Logistics could be better organized with usually larger space for arrangement off-site, and standardized components can be delivered to site directly minimizing site processing (Plate 3). The reduced number of required site labours could also improve the site environment to less congested and more organized. A major minimized site

work is welding, which is associated with eye damage, electrocution, burns and sometimes working-at-height. The replaced bolting connections eliminate or minimize the time of exposure to these risks. Modularization aims at fabricated in provisions, simplifying on-site lifting and handling. With predesigned elements, risk management planning is at a much earlier stage in a project rather than ad hoc solutions have to be delivered on-site. With elements being designed once and replicated from project to project, design can be in a more fit-for-purpose way enabling more efficient long term maintenance. For example, easily integrated edge protection provides early set-up safety net and the solution is ready whenever the system is used.



Plate 3: Off-site fabrication with organized logistics

2.4 Challenges

There are of course challenges and obstacles during the application of shoring modularization. Solutions are available and the overall benefits of pursuing modularization are not offset. Modularization involves considerable investment at the beginning on designs and establishment of production line. However, once the procedures are set in the right direction, benefits can be foreseen and become more obvious along with time. Strategic planning is essential to maximize these benefits, not only on single project but projects over a period of time. As long as the modularization design allows enough flexibility and suits a sustainable type of projects, the initial investments are worthwhile.

In between projects, there are challenges to meet the target of reusing the modules. The structures themselves deteriorate after use and storage of them in project gaps are of concerns. The recent design of modules aims at maintaining good conditions during use, for example, painted on purpose against rusting. The transferal programme of modules between projects are also planned ahead and being taken into account at tender stage, looking for minimizing storage of unused modules.

In individual project, the characteristics of the each site such as geometry irregularity and variation of span and depth of excavation could make universal applicable modules difficult to be designed. Generally, slightly over-sized members are not a problem as the modules are reusable, in member size selection this has been considered. To cope with irregular shapes, specific modules are also fabricated in minimal extent to fit in. Even some of them may not be reused, the connections are still made compatible in the modular system and materials can be recycled.

Technologies, especially digitalization, also help a lot in tackling the problems encountered. Digitalized design environment allows early planning such as clash detection and reduces the sacrifice of modules when clashing with permanent structures by changing the arrangement of different modules. It also help in terms of overall construction delivery.

3 DIGITALIZATION

3.1 BIM

BIM helps to collaborate existing and planning, spatial and sequential information along the whole construction process. This makes planning at early stage a lot easier and more effectively digs out potential problems before construction stage. In application for a deep excavation with basement construction, visualization of existing structures, permanent structures and temporary supports to be built is available. With the full member sizes illustrated, clash detection in a 3D environment before construction can be carried out (Plate 4). The arrangement of temporary works can then be adjusted to minimize clashes. In case of inevitable clashes with use of modular components, the possibly sacrificial parts to be left in permanent structures can be shortened to reduce wastage.

During construction, not only the general spatial relationships between structures are important but the detailed assembly between components may have problems, particularly with modularization. The structural assembly is rehearsed with the use of model-based construction software. The 3D modeling including the connection details enables virtual assembly for constructability study and fabrication. The construction service department who manages the steel works prepare the shop drawings and input to the environment for analysis. Early identification of assembling problems is then possible and these can be easily communicated with fabrication factory for adjustments. Any potential constraints are visualized and potential solutions can be tested in the model.

Some other applications of BIM digitalization include input and output of data, and update of site information for raising awareness and easy communication. Where adjacent sensitive structures are present, the construction works in relation to the structures can be visualized and raise caution when the relevant works are in progress. 3D printing of the digitalized models for presentation and communication becomes easy with the updated BIM in place. The model is also used to record the as-built situation on regular basis ensuring the information is up-to-date and providing an easy and complete way to produce as-built records.

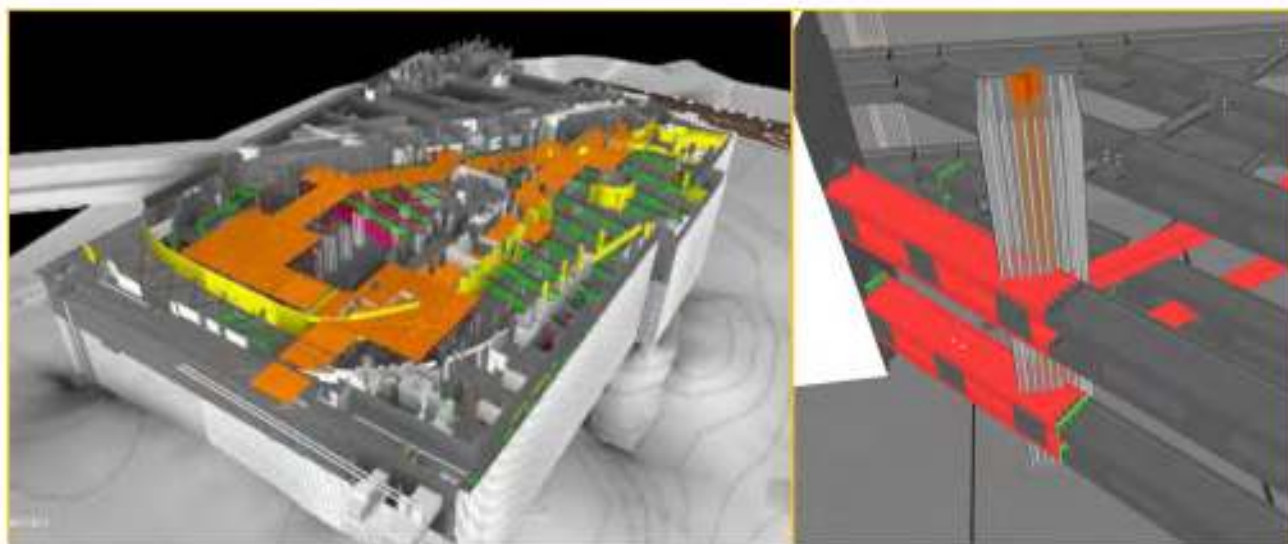


Plate 4: BIM model and clash detection

3.2 Design model and automation

With the well developed BIM information, the conversion of relevant elements into full 3D structural model is practiced in temporary works design. The structural interactions of the temporary working platform and shoring members are built in 3D geometries (Plate 5), load combinations are coupled for member checking to have a more complete picture than designing in 2D. The link between BIM and structural models also allows model based checking automation to be aimed, where when design is changed, the 3D BIM and structural model are updated accordingly. The updated structural member

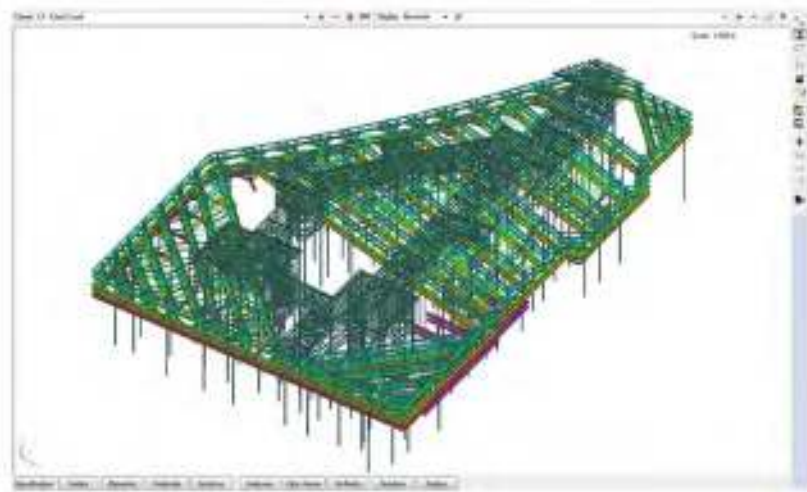


Plate 5: Full 3D structural model

checking can then be reflected right away when the same format of results are output from the model to verify the feasibility of design change.

3.3 Drone and Photogrammetry

Drones are sent to take aerial photos for site progress tracking on regular basis. With current capacity of photogrammetry models, the photos can be used to build a reasonably accurate 3D models forming point clouds and surfaces for measurements (Plate 6). The excavation volume of the site is therefore checked by comparing the differences between photogrammetry models generated at different time. An approximately 2% volume difference from the actual survey is recorded with the photogrammetric method.

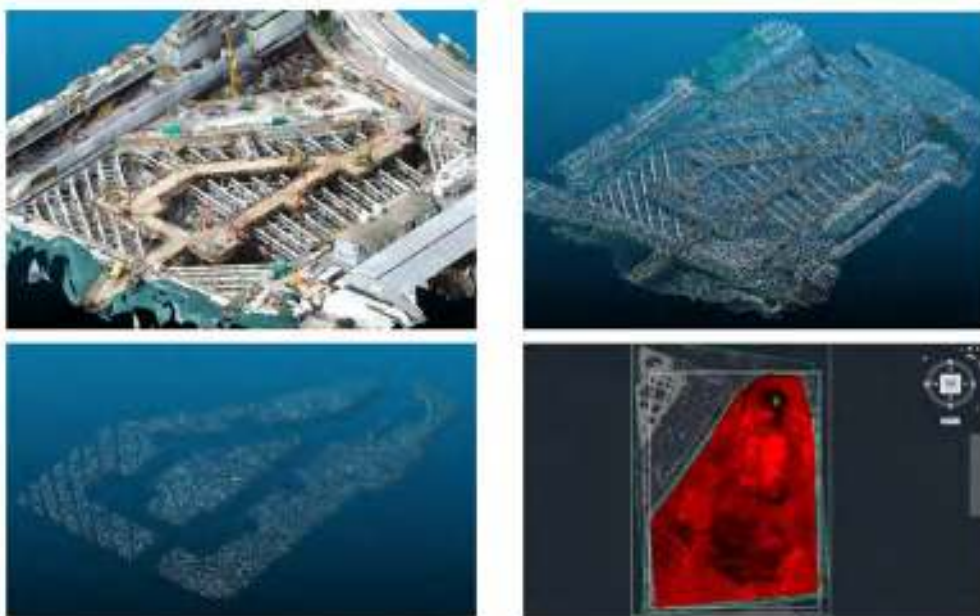


Plate 6: Photogrammetry model and excavation tracking

3.4 Project management

Resource management in a large project site is a challenge. With shoring modularization adopted, the tracking of components from factory, delivery to site, to storage and installation on site is essential. QR code on each module is used for the ease of the management (Plate 7). Each working personnel

can identify and read information about the components easily with hand held devices. The check-in of information can also be uploaded to the system for overall planning and management.



Plate 7: Coded resources management

With all the on-site information and data collected, information management can be a difficulty with loose hand written records or individual data files. A dashboard system is established to compile the information and presented in a neat way for the ease of understanding and planning (Plate 8). This can be applied to the modular ELS system, as well as labour, plant management and overall productivity.



Plate 8: Modular ELS management dashboard

4 MONITORING SENSORS

Monitoring is the necessity in a deep excavation project. Traditional monitoring devices sometimes give rise to accessibility problems and being vulnerable to damage with the connecting wires under a dynamic site environment. Wireless sensors are therefore promoted in the subject example project. Wireless monitoring instruments, compared to wired ones, are easy to install, not as easily damaged and can provide stable and reliable data for a longer duration of more than a year. They are suitable for providing sufficient data with confidence to support the verification of the overall ground conditions and performance of ELS system. In the deep excavation project here, the struts are as long as 150m in

the wide site. Wireless strain gauges (Plate 9) can be installed anywhere along the strut allowing the choice of most suitable locations without worrying about the long wiring or accessibility after further excavated down. The construction activities in the congested area which can easily damage the wires are of less concern. The time to take readings is saved and the frequency of readings can increase to hourly or more frequent.



Plate 9: Wireless monitoring sensors

With the benefits of the wireless sensors used for strain gauges, high quality monitoring data is available for close monitoring and design verification, hence enhancing safety assurance. Real-time strut forces monitoring can be carried out to track the actual structural utilization at different stages of excavation compared with the design values. Minor variation due to environmental effects such as temperature can also be observed (Plate 10). In case the forces go beyond the designed values, review of design can be carried out with the early alarm. This may allow further consideration of adopting observational method with high level of confidence in monitoring results.

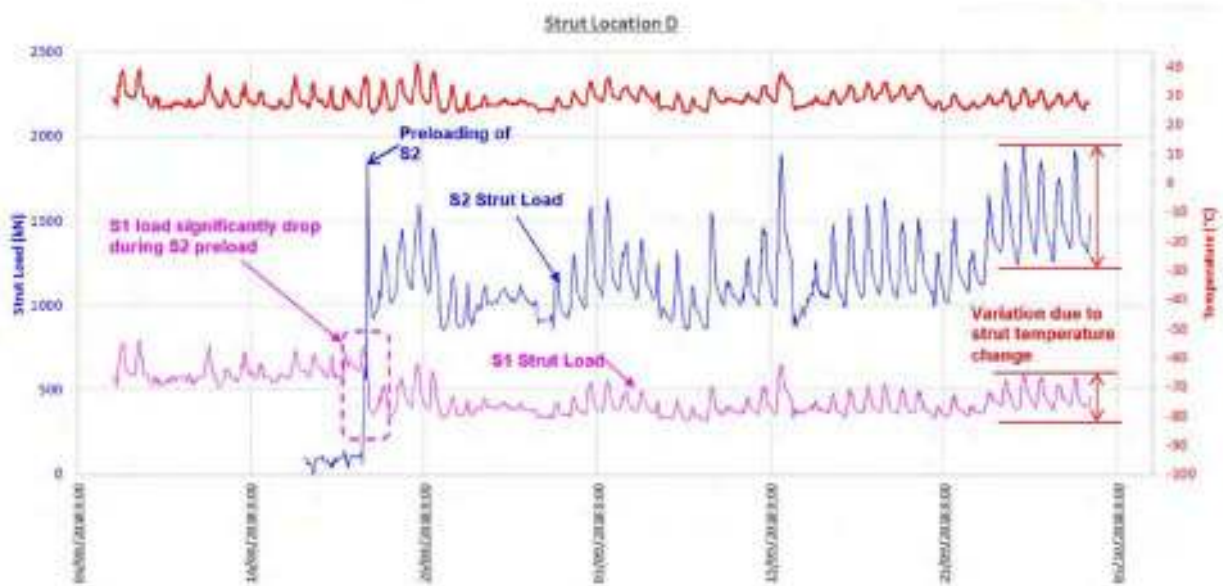


Plate10: Strut load monitoring

A new type of smart sensors is also tested in the site (Plate 11), which offers machine learning artificial intelligence (AI) capabilities enabling it to be self-calibrating and capable of optimizing data with sensors communication as well as achieving high levels of accuracy lasting for years. They are wireless sensors which form a mesh network and data can be relayed between each sensor. A family of sensors in a particular area is hence formed while line of sight between sensors are not required. A base station supporting a family of sensors allows connectivity to the internet or local network, thus the data collected by the sensors can be remotely accessed real-time. An AI algorithm is used by the sensors to perform onboard calculations, deriving real-time acceleration, inclination and displacements without human intervention (Soga 2016). Adaptive data collection and local processing of data for the extraction of failure signals are allowed. The data communications within the sensor network can also be optimized according to environmental conditions, possibly improving accuracy. The Internet of things (IoT) sensor output opens up possibilities in the future mode of monitoring.



Plate 11: Wireless smart sensor network with AI capabilities

5 INTEGRATED PROJECT DELIVERY

With the start of modularization aided by BIM, monitoring sensors and more digital tools, project delivery integrating various digital tools as a whole is eventually pursued. It is an approach for managing construction projects that fully integrates processes and people over the entire course of a project, from the initial planning and design stages through to facilities management. All processes of the project lifecycle are to be integrated and digitalized from design, planning, quantity surveying, manufacturing, logistics, construction, assembly, supervision, inspection & commissioning, handover and asset maintenance, through the use of technologies. Data is placed on cloud platform and shared with the participants in the supply chain. This has greatly improved the speed, quality and flexibility of the processes.

6 CONCLUSIONS

Along with years of development, use of standardization, BIM and sensors are becoming no more innovations but a norm. As technology advances, the tools with numerous options are readily available. It is the choice of suitable tools, streamlining the application workflow, and the integrated use of methods and technologies to deliver projects efficiently which should be our focus. While improvements in each aspect shall be looked into, for example, robotics and automation in manufacturing have great potential to be further developed. A more and more sophisticated and integrated system is looked forward to.

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Digitization of Geological Data for Clearer Understanding and Enhanced Productivity

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ABSTRACT

Ground investigation is a major part of constructions in Hong Kong. More and more geological data are available over the years and data collection together with analysis have become ever time-consuming. Considering the cost effectiveness of ground investigation works, all data extracted from drill holes are valuable that could have provided evidence and insights for our further understanding of ground condition, especially in the urban area to facilitate construction works. Working on the digitized geological data enables efficient processing of borehole logs and laboratory testing results, which are beneficial for handling large scale projects. Under the digitized environment, it is also possible the data can be presented for a clearer understanding of the ground condition, which can effectively prevent overlooking some features in ground such as geological structures, cavities in karst formation, etc. This paper will demonstrate examples that successfully apply digitization and automation in processing of data by AGS files and visualizing the borehole data in 3D models with proposed development construction. The advantages and the way forward will also be discussed.

1 INTRODUCTION

In the past, for several decades, borehole logs prepared in the stage of ground investigation prior to most of the construction had been recorded and transferred in paper format. Logging contractors recorded the information from cores and converted their findings into readable paper format before passing to the second parties. Once the data was delivered to engineering designers, manual readings up by engineers were required for the basic geological conditions to be interpreted and considered in the geotechnical design. Technology advances which facilitates digitization – data processing by machine saves time while data analysis can include much more data. From the perspective of data mining, it is beneficial to put more information into consideration as it will statistically optimize the data model.

This paper aims to discuss the current development and the possible ways forward for digitization to be practically applied and spread in the industry. The challenges faced in the works and how they are overcome are presented.

2 DIGITIZATION OF GI DATA

2.1 AGS

Digital format of data is the common practice now and we can make good use of it to develop a digitized environment for ground models. One common type of digital format is AGS data, which is globally used for generating ground investigation reports by logging contractors. The advantages are that the data is easily transferrable with small size of files, and can be transformed to other formats. Through a computer

Digitization of GI data is not solely about the transmission format between data producers and users from ground investigation to design processes. It is more important for how the data processing is benefited by the process of digitization. There are many automated ways in reading the data in AGS file through machine. One of the simplest ways is by Macro, which is short for Macroinstruction in computer science. It is a programme rule or pattern that specifies how a certain input sequence should be mapped to a replacement output sequence according to a defined procedure. Computer reading of AGS can therefore be specified by engineers, according to some criteria and patterns which are expected and located in the data for recognition. Data extraction from AGS is generally sequenced as follows,

- Via the formatting and pattern of the data saved in AGS, machine is therefore able to instantly sort out the useful data; shortly the mechanism of automation. Automation with AGS assists the workflow in current design process, which can speed up the process and save the time and manpower of engineering professionals by computer.

Figure 1: AGS data with preliminary re-sequencing

237

2.3.1 Founding assessment

In Hong Kong, predrilling prior to construction for confirmation of the founding levels are carried out in which the amount of predrilling holes depends on the pile type as suggested in the foundation code. The rockhead becomes clearer with more intrusive drill holes. The automation of AGS sets to confirm the level which fulfills the criteria of one founding stratum itself. The output example may refer to Figure 2. When there are new raw borehole logs, the new data can be instantly and additionally incorporated to the founding stratum contours in design stage. This could reduce the time to re-assess the data and avoid human errors when reading the logs.



Figure 2: Founding stratum automated by programmed AGS reader

2.3.2 Soil stiffness estimation

Standard penetration test (SPT) is a common field test that is included in the assessment of soil properties. It is used to correlate with the Young Modulus (E) of soil empirically. In geotechnical designs like foundation piling, site formation or excavation and lateral supports (ELS), SPT plot showing the site specific soil stiffness is needed for the justification of soil parameters used in calculations or numerical analyses. Traditionally, to prepare the plot, each SPT value with the tested level requires an entry of data. Massive data is usually involved in an SPT plot against depth to meet a satisfactory sampling size for the soil in site area where the process is usually time consuming.

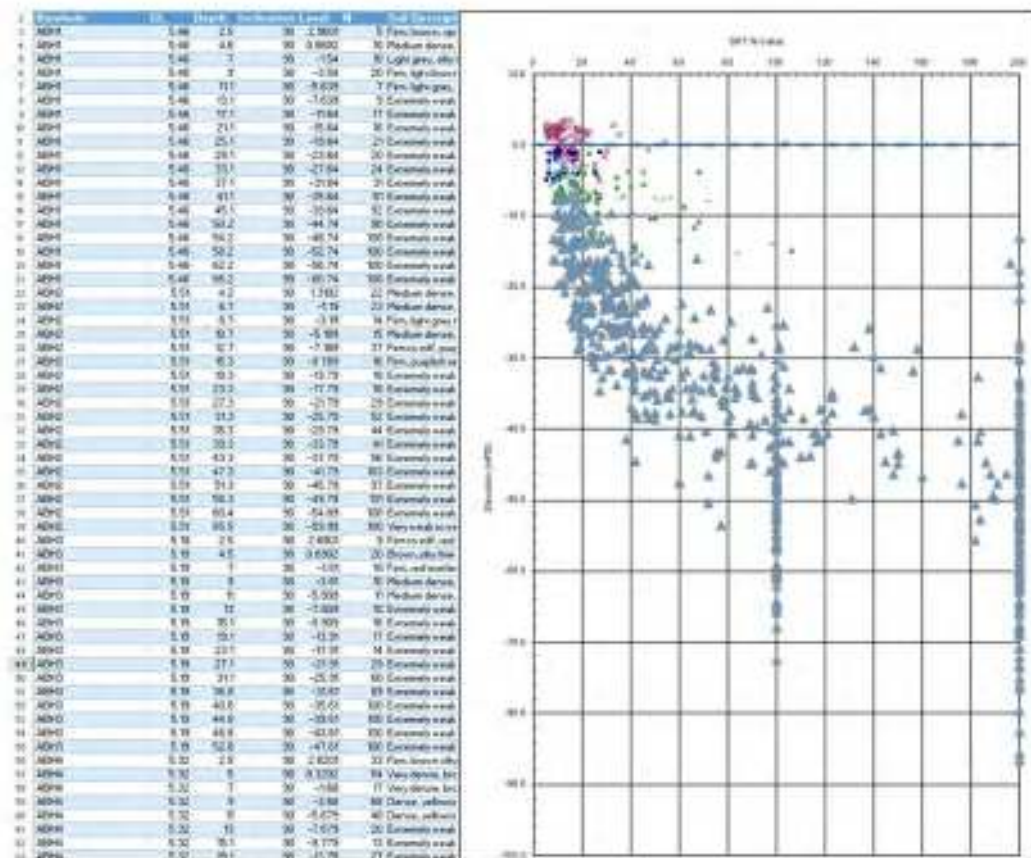


Figure 3: SPT plot prepared by programmed AGS reader

With AGS file, the values can now easily be extracted and put into the data plot; an example plot as shown in Figure 3 takes less than a minute for the computer to run. A best fit line may also be generated based on the data. Engineers are able to visualize the data and select the suitable design parameters based on engineering judgements.

2.3.3 MQD calculations

Building works in the area of marble formation require a clear understanding to the ground condition especially with the presence of cavities beneath the site. A common practice is assessing with Marble Quality Designation (MQD), which is suggested in Geo Report No. 32 (Chan, 1994). The publication suggests a calculation for every 5m section of the core in borehole logs. Large volume of data is to be processed while a formulated programme will greatly help the consolidation of the data for the assessment and presentation of the results, an example is illustrated in Figure 4. Here, MQD classes were determined from the AGS data of the borehole logs. MQD classes represented by different widths of discs to view the distribution of extent influenced by dissolution could then be quickly generated in a 3D model for visualization in BIM.

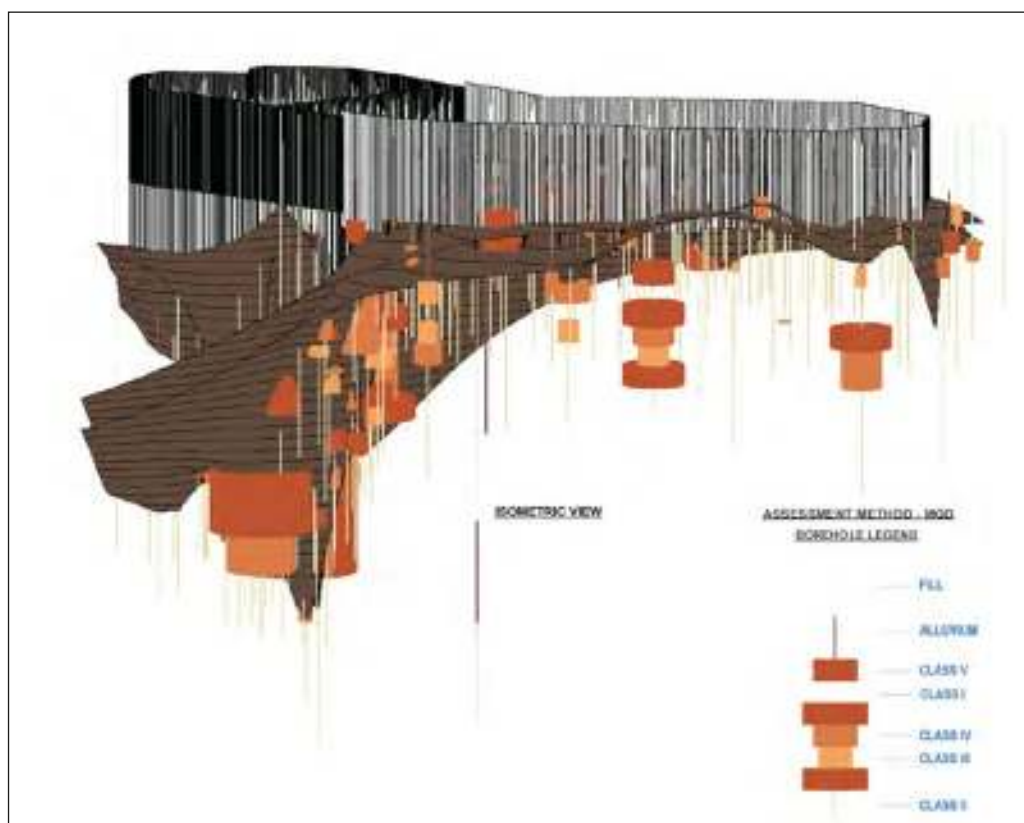


Figure 4: 3D presentation of the result of MQD calculations by formulated programme

2.4 BIM applications

In enhancing the productivity and maximize the value in facilitating the understanding from the geological information on hand, digitization of GI data could be combined with the use of analytical functions of BIM for further analyses.

2.4.1 Measurements

Quantities and cost control for construction are essential. Many items in quantity measurements and estimation relate to the site geology. For example, the pile length depends on the variable rockhead, which requires detailed intrusive ground information to confirm the level of suitable founding stratum. With the intersection tool of BIM, the 3D model will allow clear visualization of the anticipated rockhead as generated by GI data and the computer could adjust the pile lengths to hit the destined founding levels defined by the rockhead, as shown in Figure 5, to provide high accuracy and precision of length measurement.

For the cases of shallow foundation or deep excavation, the powerful and detailed measurements can provide an accurate excavation volume estimation of different materials underground accordingly.

2.4.2 Geological structure risk assessment

Geological structures usually bring impacts to the construction of underground works such as piling foundations, slope works and tunneling. Their presence commonly dramatically affects the engineering properties of material masses. The digitization of GI data is able to facilitate the transformation of data to other software for assessments on these structures. For example, useful set of data from AGS file can be extracted to BIM to provide visualization of spatial relations between the building structures and ground features in 3D models. Characterization of specified patterns automated by computer may also contribute to discover or predict the trend of some potential geological structures.

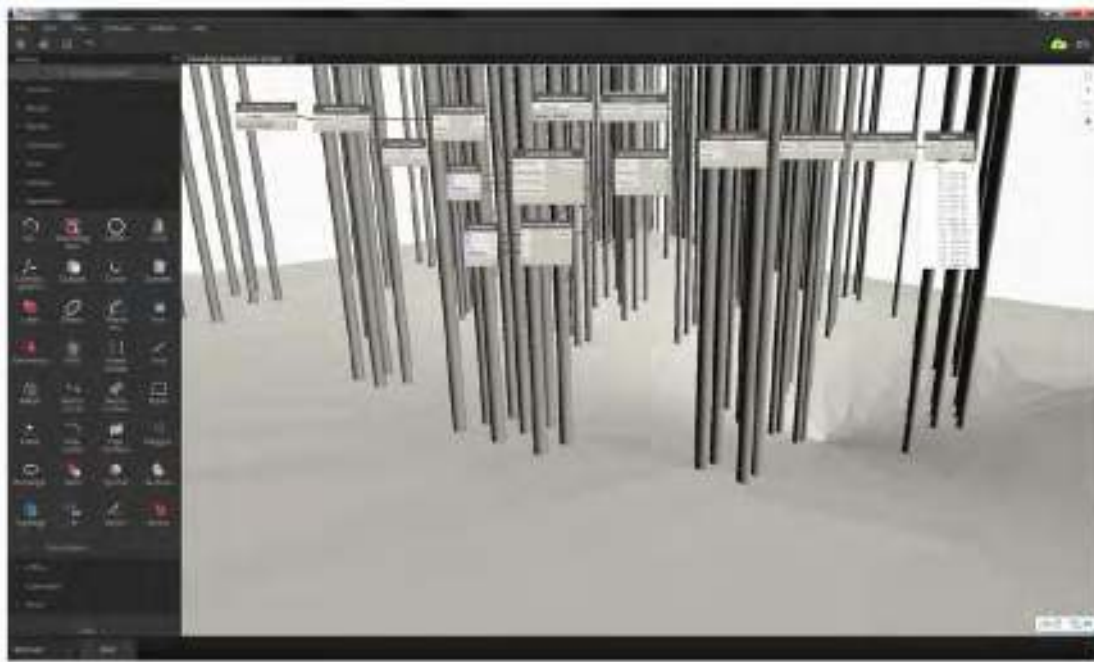


Figure 5: Rock head contour with clear visualization and measurement in BIM

Examples in Figure 6 and Figure 7 respectively present the use of visualization in BIM platform as mentioned in a foundation piling project and a tunnel project for the preliminary stage of risk assessment. The piling foundations, or tunnel structures with nearby underground utilities, are put together with the digitized borehole information. Features of geological structures identified in the borehole logs are also highlighted in the model so that the locations in the 3D environment are illustrated. Interpolation of the spot features in particular boreholes could help to predict the possible trend of the geological structures.

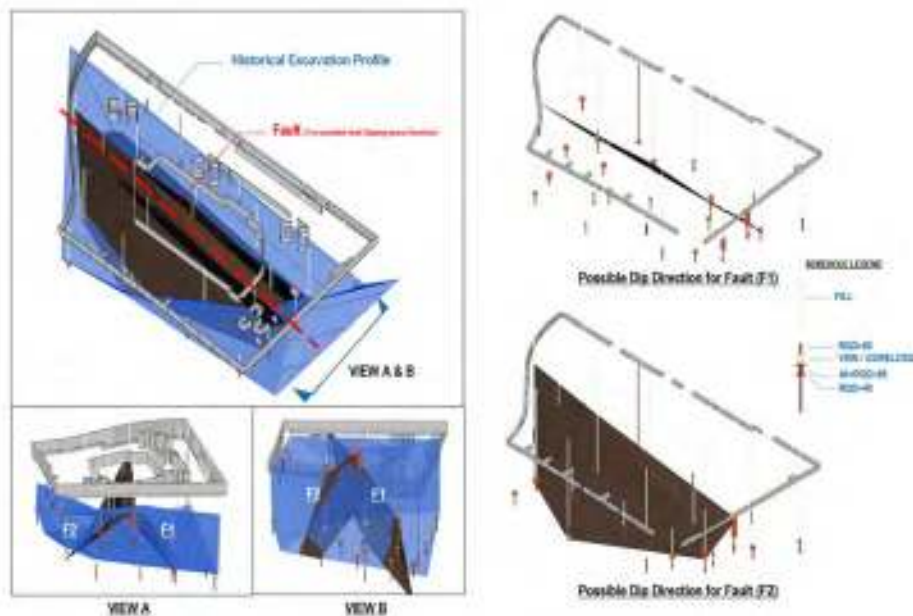


Figure 6: 3D model of the interaction between ground condition and planned construction in a foundation project

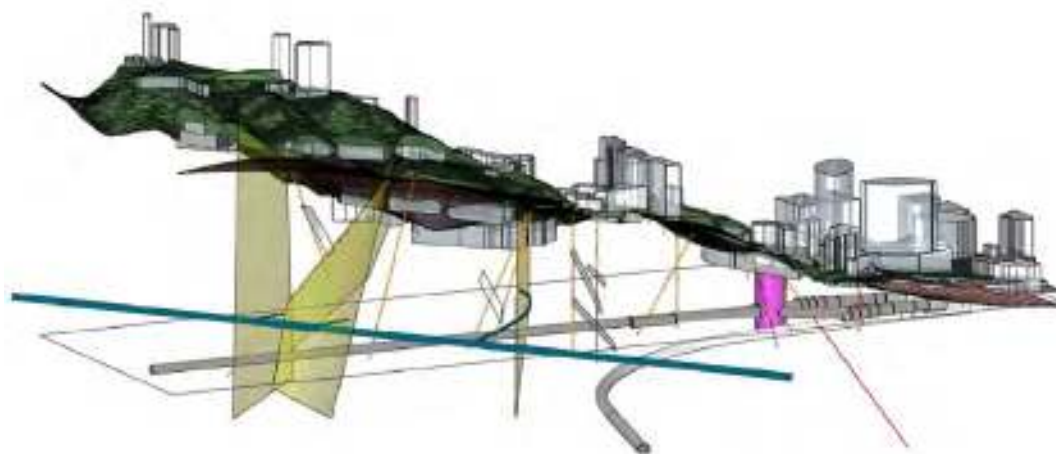


Figure 7: Projection of fault planes by 3D GI model for a tunnel project

3 CONCEPTUAL DEVELOPMENT

There are a number of possibilities worth further development which may lead to a second stage of digitization. They are being developed or researched for their feasibilities, which will be discussed in the following sections.

3.1 Parametric study

Site specific geotechnical parameters are always based on the geological data on site. In general practice of the industry, site specific ground investigations are usually carried out prior to the detailed design. Boreholes along with a series of laboratory tests are prepared and the local data is acquired for setting up the design parameters to be agreed. The sample size in statistical perspective is rather small; there are cases showing the design parameters could be very different when further predrilling and supplementary site investigations were done in the construction stage.

Ground investigation by intrusive method is of high cost. The information is valuable and important that they may provide more than expected depth of understanding of the ground conditions in urban area. With different analytical plots and data processing, some features of particular concern may be found. BIM is proficient in providing different kinds of presentations based on the input data and showing in a way that is convenient for reading. With more well developed predefined and categorized features to be looked for, presentation templates facilitating the assessment for corresponding types of work can be set up, for example, classification by soil strata, soil stiffness, rock weathering grades, or materials recovery, embedded with different parameters. Then, parameters for different classes can be modified against different assumptions or additional information to view the difference.

3.2 Geological data base system

Geological understandings to the site locality are usually progressive at the stage of construction due to the increasing number of intrusive ground information, materials exposure during the excavation, the observations on the effect of ground water table during dewatering and so forth. When the industry gets used to the practice in using BIM as the storage of project information including the geological mapping, probing records, predrilling information or even the geotechnical parameters, data could be gathered and retrieved easily when needed for future projects nearby. An example in Figure 8 shows when planning the works in a new site (Site C), a more comprehensive view may be formed with information from previous sites (Site A & B), where thickness of particular stratum (red zone) shows some connections in an alignment.

It is always useful to include more information in the geological study at the beginning of the projects which could give a more complete picture to the regional geology. Appropriate geotechnical design in light of the site specific factors can be made accordingly.

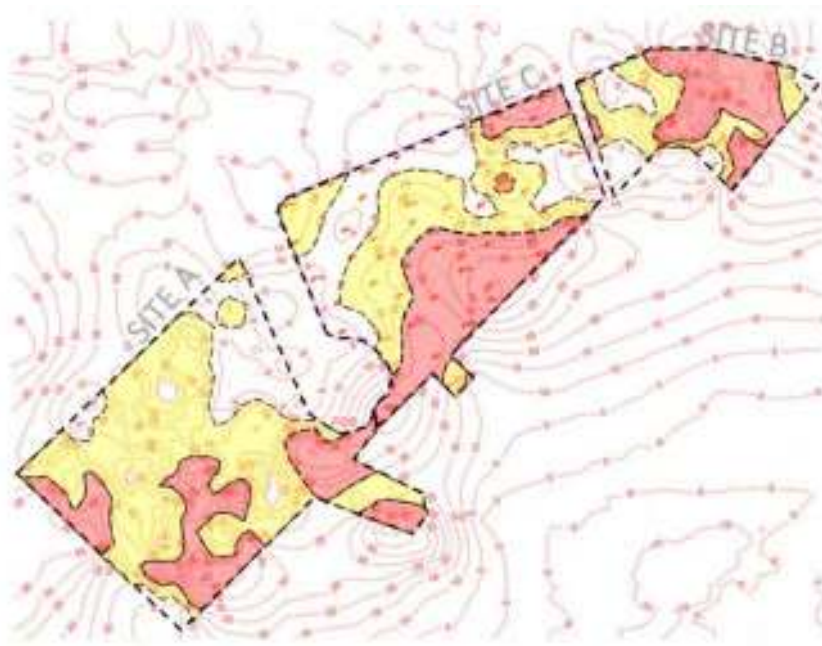


Figure 8: Incorporated GI data from previous projects (site A & B) for geological review of current project (site C)

3.3 Advice for secondary stage of ground investigation

Further to the geological data base system as mentioned in last section, some geological factors are usually exposed during bulk excavation of the site. The information is local yet it could serve as an important trace to the regional geology. When carrying out the next project in the vicinity, the observations and unclear parts in the past projects could carry over for further investigation and direct other professionals to follow up.

BIM is a powerful tool that can gather and save the information in a systematic way. It can present the data in different ways as requested by the users while it is convenient to keep updating the BIM in different stages of construction. When construction proceeds, GI data and geological findings could have been saved in the model. With the visualization of 3D model, the revealed ground conditions as well as the unknowns and uncertainties would be recorded in the BIM.

Apart from soil parameters, geological features which are equally important to the site geological condition could be saved in the BIM with a clear record and interpretation of the detailed data such as the mapping measurements, the influence zone and the recorded observations on site. As such, with a well-managed data storage system, it can form as a useful advisory framework for a more specific and in-depth site investigation for the next project nearby.

4 CONCLUSIONS

The application of machine in data processing has been moving to a new chapter especially in these few years. It is clear that digitization of GI data is not only beneficial to the efficiency but also offering a better understanding of the ground condition based on the same amount of existing information. With collaborating the use of BIM and other potential advanced programmes, more possibilities could be developed.

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How Big Data Benefits the Design of ELS in Hong Kong and the Prospects of Design Automation in the Future

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ABSTRACT

Basement and pile cap construction in the congested environment of Hong Kong is always a challenge, which requires deep excavation in confined area and excavation and lateral support (ELS) design is necessary to facilitate the construction. An ELS design involves lots of elements including assessment of ground conditions, the choice of wall type, wall embedment length as well as the shoring system. Engineers need to carry out in-depth informative and parametric studies before delivering the final design, which usually involves iterative processes. The time cost is high and the design may even fall behind the fast paced construction programme, leading to the often adoption of non-optimized design. This paper will discuss the feasibility of the use of big data in facilitating the ELS design process and reaching a more optimized design. Examples will be demonstrated for how engineers utilize the big data in the design of ELS in Gammon. The prospects of automation in ELS design will also be discussed.

1 INTRODUCTION

Deep excavation work which requires design of excavation lateral support (ELS) is already a common part of construction. This type of temporary work involves lots of iterative design check and verifications before delivering the final scheme. To assist with this heavy workload, the computer algorithm with application of the big data concept is used as a tool to provide information for the framework of an economical design or generate a design by predictive model for further optimization by engineers. This paper will first discuss the big data concept and the challenges to its application for ELS.

2 BRIEF REVIEW OF BIG DATA AND ELS

2.1 Big Data

Big data refers to the massive volume of structured and unstructured information from the existing records. To apply the currently used concept of big data, it is more about the extraction of useful information from the large data set in specific domains. Companies used to save their previous works in paper records or other dispersed variable formats; such variety of data are pure records and stored in a safe place that no one will be able to access unless it is wanted. Their values are minimal unless they are able to be reassessed and referenced. The concept of big data is to apply the analytic method and visualization to the data. In the engineering professional, this is preferably applied to get insights from the past experience in terms of the pattern of data recorded.

2.2 Data Base

In forming the data base, it is usually to gather a variety of data in order to discover some useful but possibly unexpected patterns of data. There are many types of data; they could be divided into 5 groups which are enterprise data, transaction data, social media data, public data, and sensor data (Singh et al. 2015). For various sectors like banking, marketing, health care, insurance and so forth, they tend to include as much data source for data interpretation given that their targeted behavior and information to be looked for is previously unknown but potentially useful. Yet the challenges along with such large volume of data would be the data quality, data streaming and dimensionality (Singh et al. 2015). It is hard to handle with the high complexity of data. With time changing, the data source in different phases may be free from control and guidance of anyone. The tool to store and analyze the data would be highly demanding.

Considering the engineering aspect, one design includes lots of assumptions that may not be always applicable to one region or one company's approach. Only part of the enterprise data would be suitable for the data base. In addition it is of high cost to develop on the data mining from the wide variety of data and sort out small part of data from less irrelevant data. Gammon was therefore turning to set up a specific data base by developing the new ELS design platform and commencing to collect the design data from latest projects. One advantage is that the experience reflected by data complies with the updated approval requirements and code of practice. They are buildable and company specific. Another advantage is once the platform is set up and all the design is carried out in the particular format, it can save resources especially the professional manpower in data collection and mining from old records. As a result, with this approach the data base can easily be centralized for valid data retrieval.

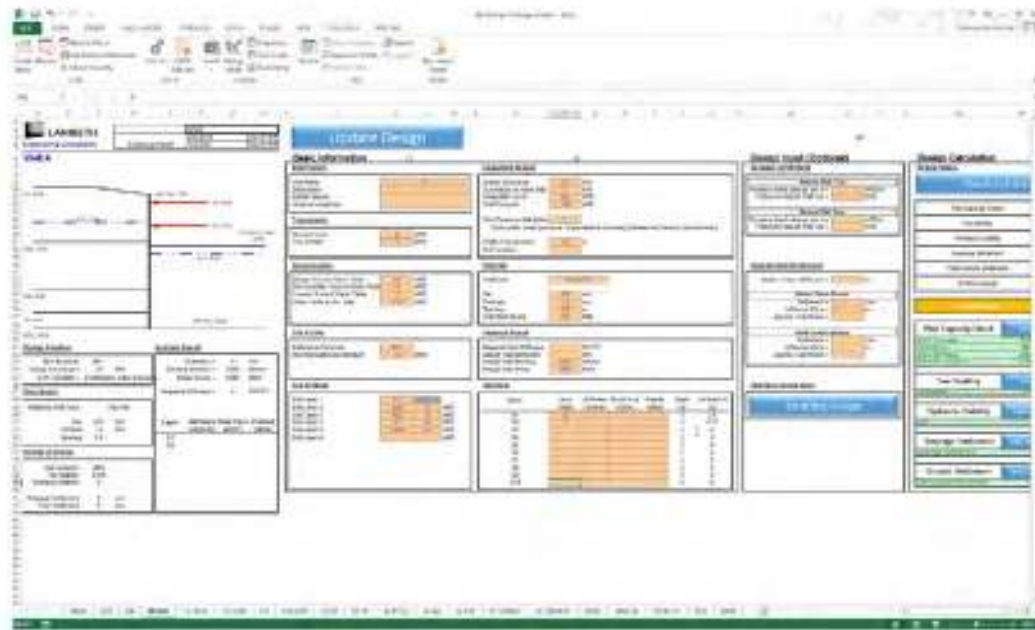


Figure 1: Design in particular format on ELS Design Platform

2.3 Knowledge Discovery Process

The goal to set up the data base is to discover the knowledge which turns out to benefit the concerned business. The target knowledge is known to be some trends and patterns in the data, which requires some processes to the collected raw data and analyses on the sorted dataset. Upon the data base accumulates sufficient amount of data, a series of knowledge discovery process as summarized in Figure 2 can be carried out. Data selection is the first step to select the valid ones and eliminate those with error. Data pre-processing includes a new format of the raw data which is prepared for the next step of data transformation. This could involve the data to fit into the analytic tool and programme. Data mining will be carried out when the suitable and sufficient amount of data is ready. Hopefully the knowledge and useful patterns of data could be generated in the last step of data interpretation and evaluation.

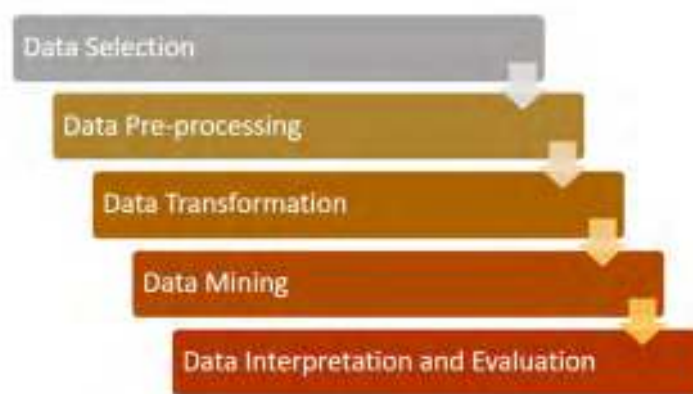


Figure 2: General Discovery Process on Data (Singh et al. 2015)

Initially a similar way of utilizing the past experience for ELS reference was putting the total strut and waling tonnage from different project against average excavation depth into a graph as shown in Figure 3. Though there is not a clear proportional relationship between two values, there is a general trend with larger variation which provides a rough and early idea of the potential application of big data concept to ELS. With the current application of the big data concept to ELS design, the variation of data is far lower since the types of external factors in the design consideration are few. From Gammon's perspective, the value to keep the standard and integrity of works is important, we therefore opted for setting up the in-house data base as data selection and data pre-processing. Data transformation can then easily be done with the ELS design platform aforementioned. Data mining, interpretation and evaluation are the harvesting processes and will be discussed below.

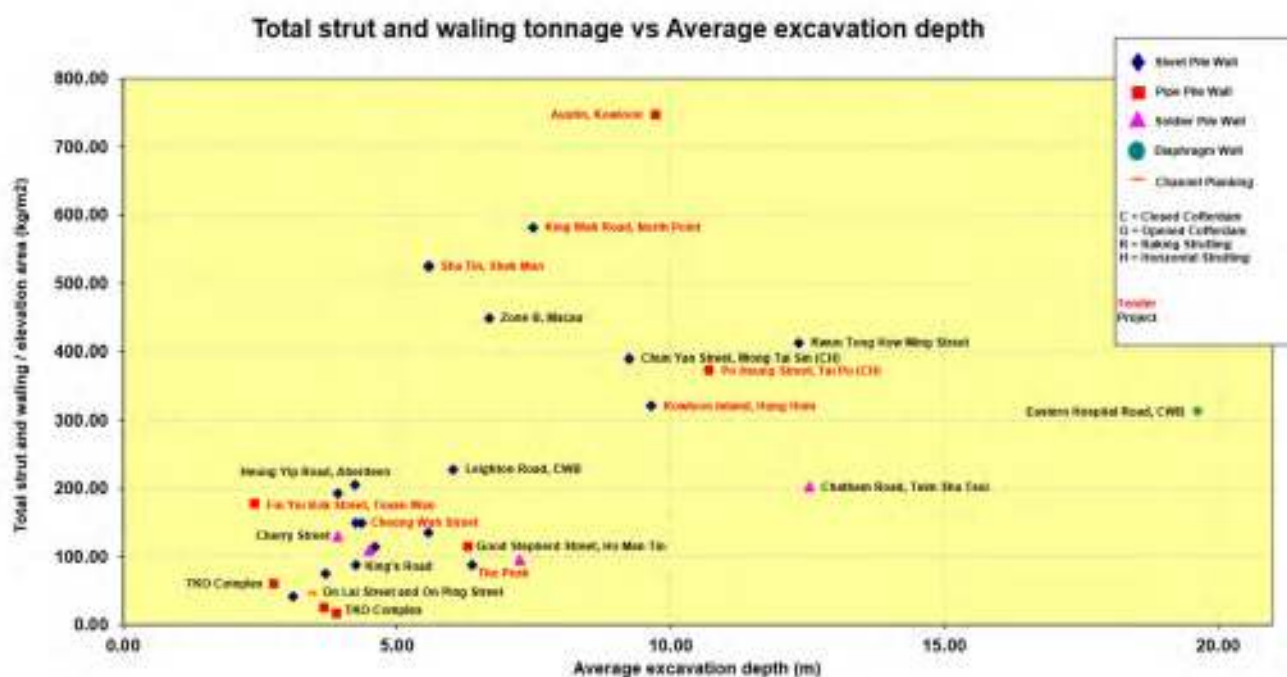


Figure 3: Completed wall design data plot

3 DATA BASE AND PREDICTIVE MODEL

3.1 ELS Data Base

As mentioned in previous section, a relatively small but specific sampling cluster approach is adopted; the data mining is able to focus more on the concerned engineering perspectives without heavy data selection and pre-processing works. Excavation and lateral support (ELS) is temporary work aiming

at replacing the excavated soil volume and support the active pressure from soil, underground water and surcharge. With the new format of design platform, design information is centralized in unit of wall design section and saved into the data base for later data analysis. The database has kept expanding in increasing rate once the design platform was set up as illustrated in Figure 4.



Figure 4: Feedback System for the ELS Design Work Flow

Lambeth Associates Limited as the in-house consultant of Gammon Construction Limited has made use of the ELS design experience of the foundation jobs in Hong Kong geotechnical and geological settings over the years to form a data base. With the advantages of high frequency in providing design service to Gammon's tender and project, design data is accumulated in an elevated rate after the set-up of standardized design format and platform. Sufficient amount of as-built design data has shown some patterns and correlations between different settings and design products were uncovered.

A common work flow is when a new potential project is received by the estimating department for tendering and involves the in-house designer consultant for ELS conforming design check and exploring alternative value-engineering scheme, the data base will record the data. The design is in the process of optimization at this stage. After the tender submission and contract awarded, the design will be further modified during approval from authority and construction stage, which incorporated the constructability and approval consideration. With the continuous updates to the design during the processes, the data base will increase its sample size and the overall data trend will be shifted to have considered more design cases thanks to the feedback. Basically each wall design section involves three general categories of data that could contribute to the data base. When one design is completed and the items are input, the corresponding data base will be updated automatically. They are listed in Table 1 as below.

Table 1: Categories of ELS data

Categories of ELS data	Soil Properties / Ground Water Condition	Vertical Elements	Horizontal Elements
Items	Soil Parameters (e.g. Young Modulus, Soil Shear Strength)	Wall Type	Sturt Member and effective stiffness
		Embedment Depth	Strut Layer
	Soil Profile & Retained Water Depth	Retained Height	Preloading Force

3.2 Analytical Method and Trend

Every wall section in the design is unique and independent. Engineers used to design one by one for optimizing scheme. The first step starts by carrying out the site investigation based on the received ground information including the boreholes logs and the laboratory test for the soil properties. With

the raw data within the site area, a specific set of soil parameters could be set up for the later stage of checking. After preparing the layout of ELS, a division of the whole project ELS into separated wall types would be done. Geotechnical models would be set up in order to analyze the force and movement of the elements. Each wall type design has its own individual design and model outputs. The designs comprising the three general categories of data as mentioned in last section from different projects over the year have been being collected for analyses as shown in Figure 5.

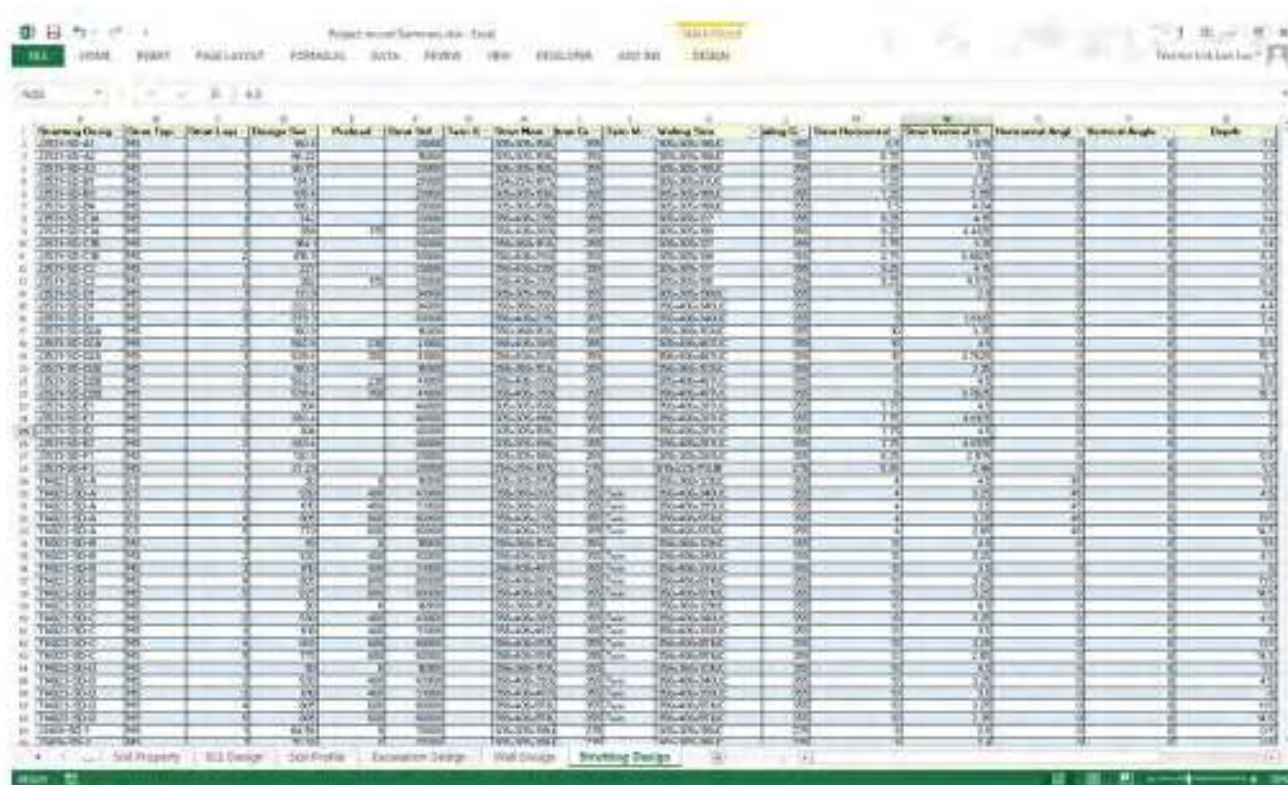


Figure 5: Completed wall design data collection

The analytical method is set to study the behavior and the forces anticipated in the wall and struts with certain soil profile of particular soil parameters. When large enough volume of data is stored, a range of value relationship can be established by computer. Though this cannot be correlated and simulated with a complicated mathematical formula for the behavior and forces, a trend in the dataset can be generated, which is the useful information to be dug from the data base, shortly the concept of “Big Data”.

When the data base is there, meaningful trends are looked for. Examples are as shown in Figure 6 and Figure 7, respectively showing a linear relation between the main strut tonnage and excavation depth, and a quadratic relation between the main strut tonnage and design soil load. Although the data are all individual design with site specific geotechnical conditions, they do align with certain patterns which show the design cases with similar excavation depth / design soil load do require a similar tonnage of shoring supports.

3.3 Predictive Model

Predictive model is the outcome by the machine learning algorithm. With the input of design situation and the pattern interpreted from the growing ELS data base, ELS design prediction for reference can be generated. The mechanism is illustrated in Figure 8.

The main purpose of the ELS predictive model (Figure 9) is to provide a very quick ELS tonnage estimation for early planning. Meanwhile, engineers can review the estimation and further adjust particular ELS elements to optimize the design. Since the predictive model is fully generated based on the ELS data base collected from past experience in the company, they are reliable enough as the

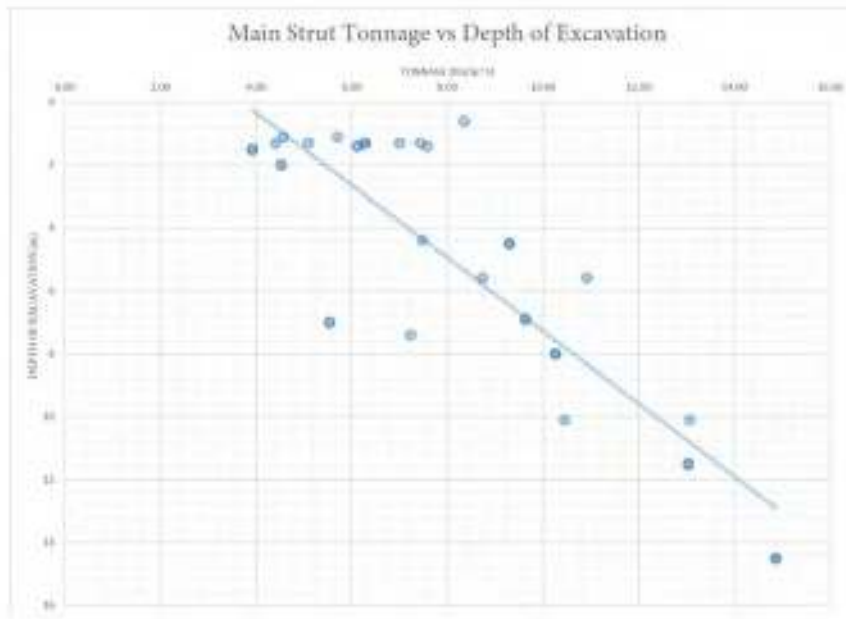


Figure 6: Example trend from data set (main strut tonnage versus excavation depth)

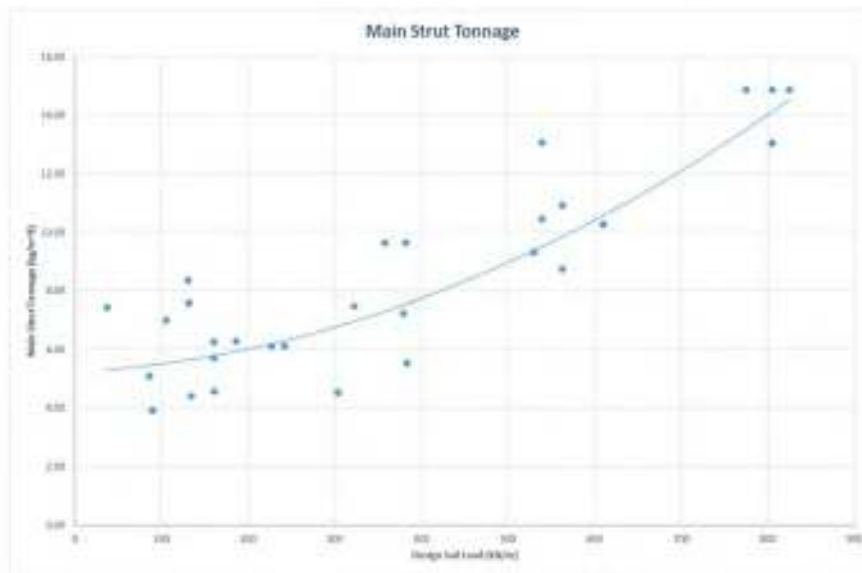


Figure 7: Example trend from data set (main strut tonnage versus design soil load)

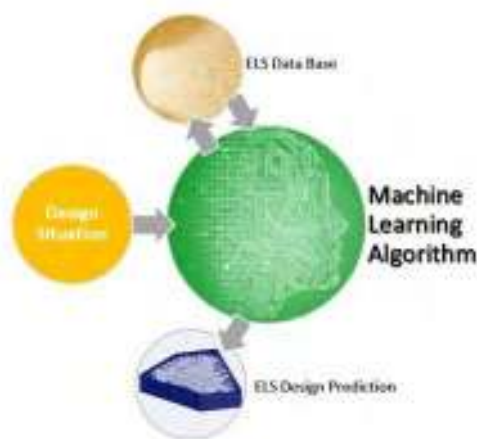


Figure 8: ELS Design Prediction Model Mechanism



A worked example is demonstrated in Figure 10a & Figure 10b, where when design in construction stage was completed, it showed a very similar outcome with the predictive model including the number and levels of shoring, and the embedment depth of wall. With such predictive model, engineers have a rough framework of design with ELS shoring tonnages. The sequence is quick and simple which benefits the work efficiency; general procedure is listed below.

1. Consolidate the design situation of excavation and review the site geology
2. Input the design situation to the model
3. Select the job information from the database with similar retaining structure and geological conditions
4. Design the shoring layout and estimate the strut length and load width
5. Generate the model
6. Review the constructability of the predictive model outcome based on the site constraints
7. Form the design with reference to the predictive framework

Users though should remain independent in delivering the design that the model is a kind of integrated information in assisting the design. It provides a rough framework that the new design may be built within the suggested economical tonnage while engineering judgements are still very important in different stages for double checking and screening out invalid information provided by the prediction model. For new ELS designers, it may also serve as a guidance to form a new design with a fair mindset of economical design, however, they should revert to proper advice from literatures and theories for uncertain parts of information. After all, the engineering principles and judgement are not superseded but supported by an experience gaining tool.

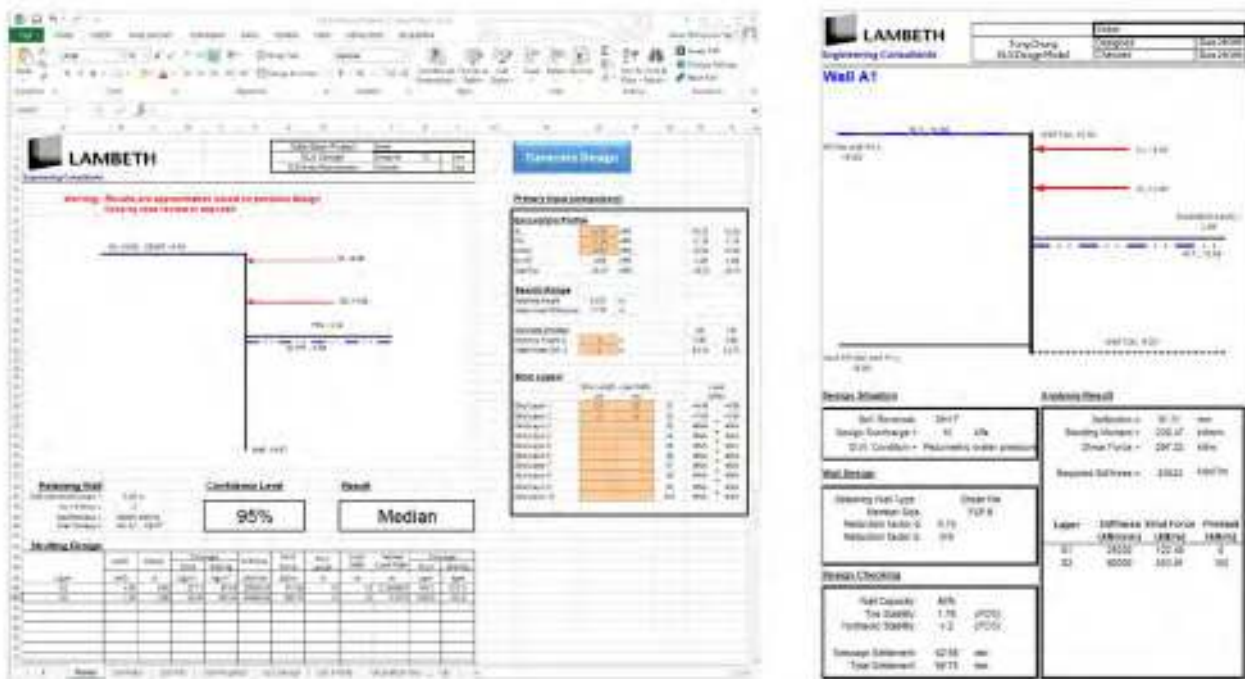


Figure 10a (left) & 10b (right): Outcome from Predictive Model and Construction Stage Respectively

4 AUTOMATION

Automation will be the way forward when the efficiency to the work flow is the main concern. As long as some works are replaced by the automated work flow, the manpower and resources can be shifted away for digging into other form of opportunities. The automation would be applied with cross reference to software model, likely to be BIM in the foreseeable application. BIM could have already stored the individual project data. The transferrable format will be driven by spreadsheet at the beginning of development while it could have involved lots of calculations, spreadsheet and graphical empirical method. The following discussion will be the potential and developing aspects of automation.

4.1 Parametric Studies

Setting up the parameters for site geological condition is always the first step for ELS design. They would be based on the values from ground investigation data in advance of job submission stage including the boreholes logs, field tests and laboratory tests. Sometimes due to the very short timeslots and fast paced submission period, only the local ground data have been considered and the ELS design will commence. This could be questionable if the sample size of data is sufficient enough to support and represent the properties of ground condition, or if the regional geology is overlooked. With the data base to store with different previous job references by region, soil parameters may be more comprehensive and statistically accurate, illustrated in the example as following Figure 11 comparing smaller sample size and bigger data.

4.2 ELS Shoring Layout

With the benefit of feedback system, the predictive model will become ever more accurate and reliable in the future. Engineers may focus on reviewing the design and fewer modifications and variations from the predictive model will be expected, raising the pace of design process. More and more new projects in Gammon have been adopting the modular strutting, which simplifies the shoring arrangement by standardization. Some typical shoring arrangements by modular struts are formed to suit different forms of cofferdam and cater for different site constraints.

Automated shoring generator will be the next stage of the prediction model, which aims at being formed in BIM itself. With the typical designs and modular struts, options of ELS shoring layout can be

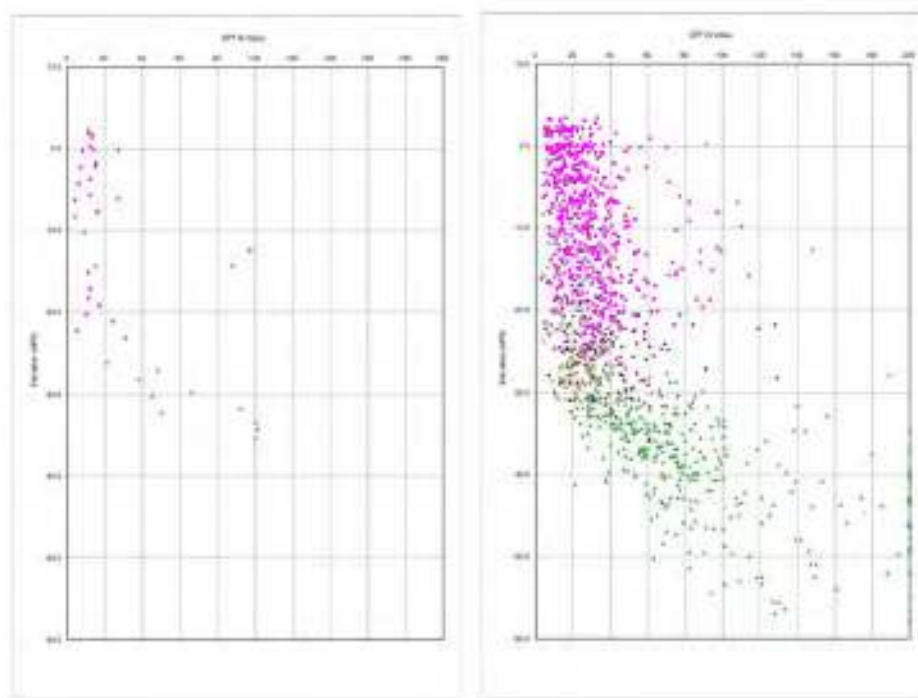


Figure 11: Difference in soil parameters with small data and big data around the same site

generated in the framework of prediction model of the project. Engineers could only select the optimal shoring scheme with the most reasonable tonnage and arrangement among all the proposed schemes and review the constructability.

5 CONCLUSIONS

The big data concept should be able to benefit different sectors of professional as long as the technical challenges like implementation and visualizations can be well handled in the future. In ELS design, though there are many geotechnical considerations in technical perspective, patterns between certain conditions and the required design settings do exist in the data base of existing successful designs, and interpretations can be made to form predictions. With such information, the design of ELS will be greatly helped and the optimization of design is more probable. Automation is the next step forward that should be further developed. Professionals with the assistance by advanced technology will soon change the time allocation in conventional design process, getting away from routine data management and iterative works to more focusing on the engineering judgement, constructability, safety and sustainability of the design.

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Digital Application in Foundation and ELS Designs

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ABSTRACT

The increasing complexity of projects in the construction industry contributes to stimulate the development of information technologies to support their implementation. As an engineer, how can we streamline the design process to improve the efficiency and enhance the competitiveness? In the project development, it is important to provide accurate and reliable information and it is also necessary to establish good digital communication platforms for coordination of design, planning and surveying.

The adoption of Building Information Modeling (BIM) methodology brings benefits to the foundation projects through technical improvements, reducing the duration of tasks and increasing their efficiency. These aspects, in addition to improving the transparency and quality of the projects, reduce costs, thus could make the geotechnical engineering industry more competitive.

This paper presents the past results and techniques adopted using BIM in tender and construction stages for various foundation and Excavation and Lateral Support (ELS) projects. Some key processes of design will be discussed.

1 INTRODUCTION

The digital application in engineering practice poses challenges to foundation and ELS designers as the trend of construction industry demands greater efficiencies and competencies with digital technologies. Like decades ago when using computer programmes was started, i.e. CAD, structural and geotechnical software, to replace hand sketch drawings and hand calculations, the role of computers in engineering practice has now evolved, where Building Information Modeling (BIM) is becoming a baseline requirement for clients. However, the integration of BIM into engineering practice still requires more explorations and studies in the industry.

The main objective of this paper is presenting a key digital process by Building Information Modeling (BIM), focusing on the design of foundation and Excavation and Lateral Support (ELS) system. The application of new technologies is beneficial to foreseeing the site constraints for resolution of the engineering problems in an earlier stage. The proposed recommendations and best practices will be further discussed to promote achieving a higher degree of efficiency.

2 BIM APPLICATION PROCESS

BIM brings the construction industry into the digital age by creating and using a completely 3D digital representation of both the physical and functional characteristics of the construction elements. This is important to help streamline the design process which improves the efficiency and enhance the competitiveness. An example of streamlined BIM design process for foundation project is shown in Figure 1. The resulting building information models become shared knowledge and resources to support the decision-making about a facility from the earliest conceptual stage, through design and construction, across the operational life eventually to demolition stage.



Figure 1: Example of Streamlined BIM Design Process for Foundation Project

2.1 Digital Data Collection

In the past, engineers often spent plenty of time to input a huge amount of data for determination of soil parameters, analysis and design. More and more companies are realizing the benefits of using image capturing software tools such as those with optical character recognition (OCR), which can help improve data processing by digitizing paper documents and eliminating the need to print some documents at all. It can simply store documents on the computer or mobile phone for later retrieval. With the support of OCR software, engineers can obtain the data in Excel format directly and use it efficiently and accurately, though it is always a good practice to verify the data prior to further use.

OCR software can convert scanned documents into text editable and searchable files (see Figure 2). However each type of OCR software offers differing capabilities. Some important factors to be considered

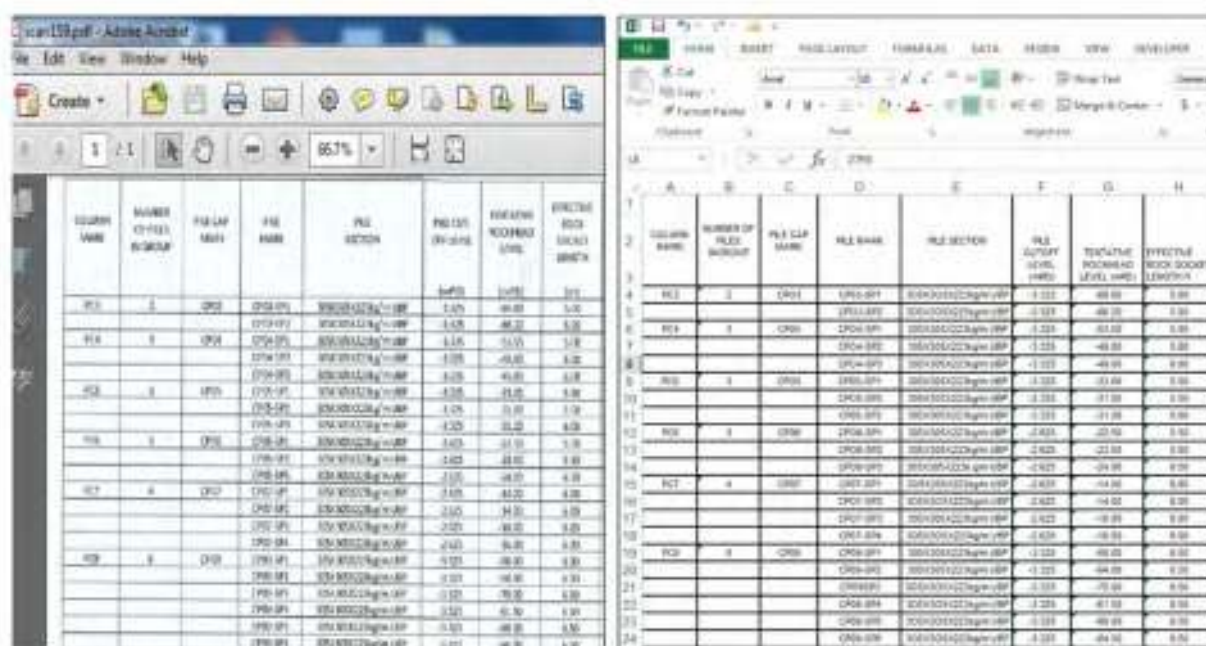


Figure 2: Scanned Schedule converted into Excel Format by OCR software

when conducting an OCR software comparison are: accuracy, speed, processing power and price. If the OCR engine is not accurate and fast, then it is no better than manual data entry. An inaccurate OCR engine will produce errors and impede the workflow. Currently, the accuracy rates of available OCR software in the market range from 60% to 99%. Processing power refers to how many documents the engine can convert at once. Price is usually the overriding factor for many companies, and is usually inversely related to the capabilities of the software. It is important to select a right OCR software to make digitizing processes faster and retrieve better information.

Adoption of the AGS Data Format shall be promoted for transfer of all geotechnical and geo-environmental data in Hong Kong by electronic means. AGS Data Format is well developed by Association of Geotechnical & Geoenvironmental Specialists. It would be beneficial to the construction industry when all parties pass on all project data by common electronic means. Examples of AGS data extraction and rockhead contour creation using SURFER are shown in Figures 3 and 4 respectively. These can be done quickly with appropriate AGS data, then the rockhead contour can also be imported into BIM model easily, where the data incorporation process in BIM is streamlined.

BH No.	Easting (m)	Northing (m)	BH Top	Top of MB	Top of All	Top of CDG	Top of CS	1/d rockhead	1/d
PD-BPC1A-P1	836508.72	819961.3	34.14			9.96	9.96	0.24	
PD-BPC1A-P2	836576.19	819971.29	32.26			24.86	24.86	12.16	
PD-BPC1A-P3	836585.65	819976.2	32.86			23.36	23.36	4.28	
PD-BPC1A-P5	836581.01	819953.36	31.92			13.92	13.92	11.92	
PD-BPC1A-P6	836587.46	819964.14	31.91			26.95	26.95	15.03	
PD-BPC1A-P7	836594.06	819976.63	31.85			27.55	27.55	5.4	
PD-BPC1B-P1	836594.38	819953.14	31.72			17.22	17.22	6.95	
PD-BPC1B-P10	836632.48	819954.89	31.64			25.56	25.56	16.35	
PD-BPC1B-P11	836634.3	819930.16	31.47			18.83	18.83	10.45	
PD-BPC1B-P12	836633.89	819946.38	31.58			17.8	17.63	14.89	
PD-BPC1B-P2	836602.29	819967.48	31.82			25.45	25.19	8.04	
PD-BPC1B-P3	836603.68	819954.55	31.85			30.93	30.93	10.11	
PD-BPC1B-P4	836602.22	819942.67	31.76			20.51	20.51	6.86	
PD-BPC1B-P5	836615.36	819965.52	31.95			29.05	29.05	7.63	
PD-BPC1B-P6	836613.28	819953.56	31.71			25.61	25.61	5.35	
PD-BPC1B-P7	836609.07	819936.67	31.71			20.02	20.02	8.74	
PD-BPC1B-P8	836620.51	819948.7	31.77			17.07	16.85	16.52	
PD-BPC1B-P9	836622.76	819938.42	31.53			14.09	14.09	14.09	
PD-BPC2-P1	836550.33	819886.07	30.68			-30.31	-34.42	-26.33	
PD-BPC2-P10	836589.09	819906.03	31.74			-7.21	-7.89	-7.89	
PD-BPC2-P11	836601.95	819901.35	31.53			-3.9	-3.9	-3.9	
PD-BPC2-P12	836601.7	819914.17	31.62			-0.88	-1.12	-1.12	
PD-BPC2-P13	836587.98	819873.89	28.71			-0.84	-0.84	-0.84	
PD-BPC2-P14	836594.02	819880.7	28.88			-8.46	-8.46	-8.46	

Figure 3: Example of AGS Data Extraction

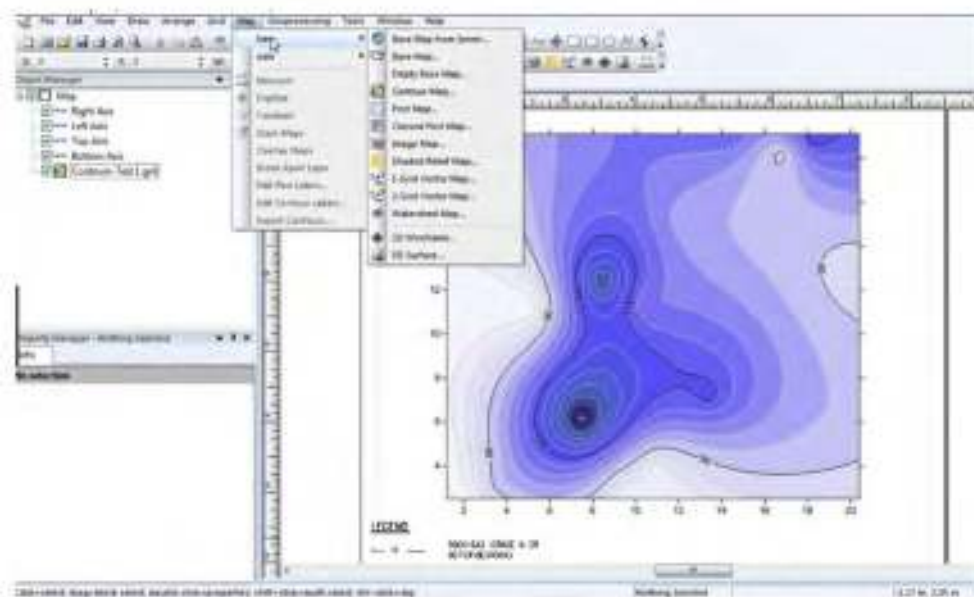


Figure 4: Example of Rockhead Contour Creation using SURFER

2.2 Design Collaboration and Communication

Taking advantage of digital tools linked to cloud, intelligent 3D BIM models allow different design teams to collaborate from anywhere in real time on the same design to communicate and inform each other of project decisions. The cloud-based model becomes a platform where survey, engineering, and construction data can all be more tightly integrated. Once integrated, the workflow can benefit from the enormous computing power.

The design engineers and draftsmen can work on the common BIM model simultaneously by setting up the worksets on BIM model (see Figure 5). The model can be edited and synchronized in the cloud from anywhere you have a connection to the internet. This allows collaboration with multidisciplinary 3D models for clash detection and construction phasing simulation, so that project teams can identify and resolve issues that might delay the projects, before the onset of construction. During construction, this improves communication, and reviews with access to project information anytime and anywhere are available.

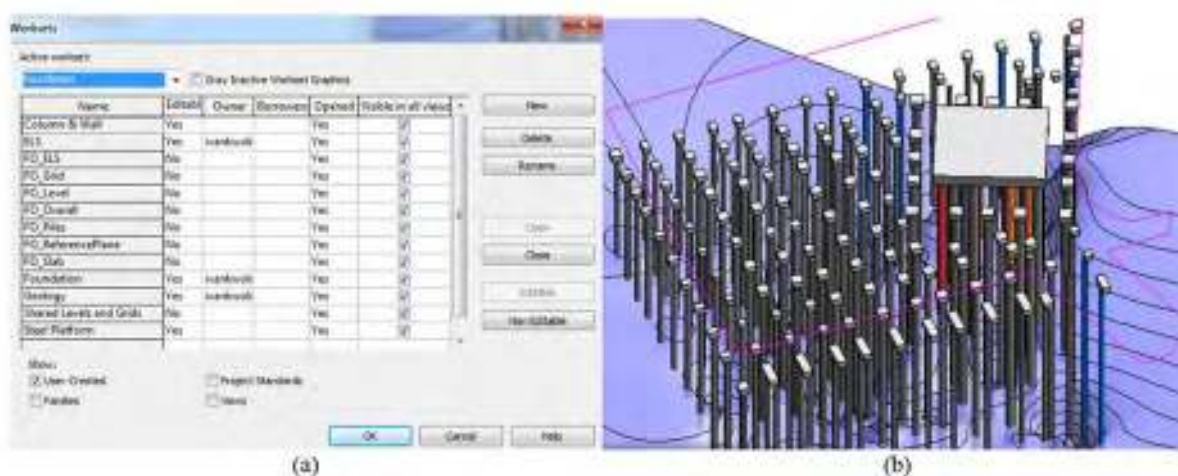


Figure 5: Example of Worksets Setup on BIM Model for Managing Collaboration
(a) Creation of worksets and add elements, (b) Only the assigned user can edit the elements in a particular workset

2.3 3D Visualization

It is a big challenge to understand the site constraints and underground conditions for foundation projects. It is important to understand the ground profile information, as the production rate of excavation is affected by the type of soil/rock materials such as fill, marine deposits, corestone and bedrock. The corestone and rockhead contours can be generated by SURFER based on the drillhole records and imported to the BIM model. Engineers can then read the borehole data and be aided to reasonably foresee the related ground conditions on BIM model instantly (see Figure 6). It helps to facilitate the site feasibility studies with the crystal soil strata information, and the estimation of cut and fill volume.

The relationship between proposed foundations, sub-structures, ELS works and geological ground profiles, adjoining existing foundations, geotechnical features, sensitive structures, etc. can be visualized clearly on the BIM model (see Figure 7). For instance, it is identified the anticipated high rockhead and pipe pile is proposed by the engineer as the temporary retaining wall considering the drivability.

2.4 Clashes Detection

Site operations are highly dynamic. For example, whenever an architect makes design changes, it often comes in the form of a rough sketch that may miss important details and cause confusion. Although drawings and construction plans are there for reference, the coordination of manpower, equipment, materials and construction methods mainly relies on human and verbal communication. It may not

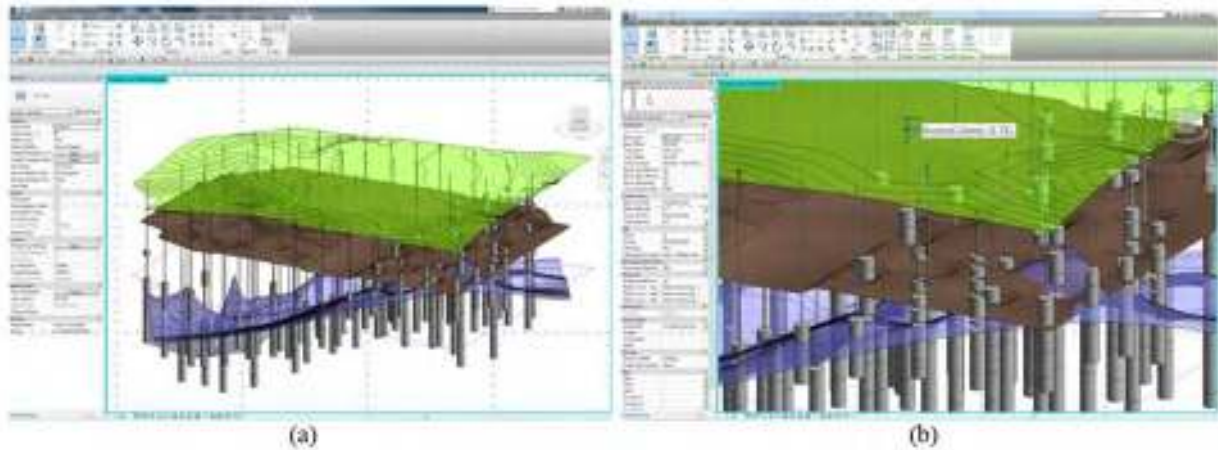


Figure 6: Example of Foreseeing Ground Profiles with Available Information
 (a) Existing Features including adjacent buildings and ground condition,
 (b) Proposed Features including pilings, pile caps, ELS struts

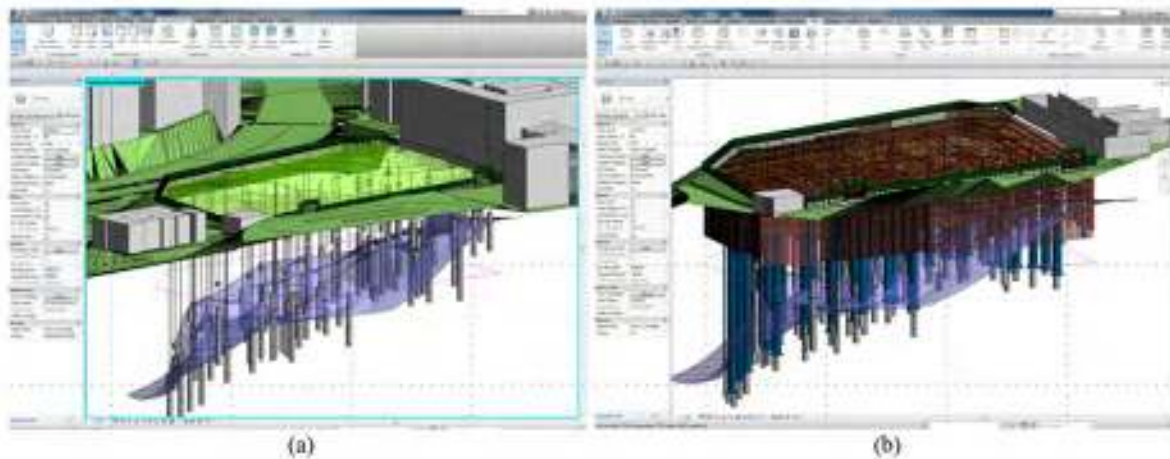


Figure 7: Example of BIM Model of Foundation Project with Existing and Proposed Features
 (a) Adjacent buildings and geological ground profiles, (b) Integrated foundation and ELS elements

reflect the real-time situations on-site. As different teams are not always on the same page, clashes and conflicts will likely happen, leading to rework after abortive work.

The clash detective function by supporting software Navisworks is a very powerful way of automating some of the design coordination processes. Clash detective can be set up to cover specific rules to check if different geometries occupy the same space in the model. At the design stage this should be straightforward to resolve and avoid the need for costly rework, which would become inevitable if the issues are not discovered until a project reaches the construction stage. To run a clash detection users need to select which files or parts of files that they want to test for clashing against each other. The selected BIM model components can be imported to Navisworks for clash detection (see Figure 8). Navisworks can then export the clash results into a summary report.

Engineers can coordinate to refine the model and eventually provide a clash-free construction solution with model-based collaboration to avoid ELS clashes with both permanent and temporary structures. It helps to minimize the rework and abortive work which consumes extra time, creates commercial disputes and increases the safety risks in construction.

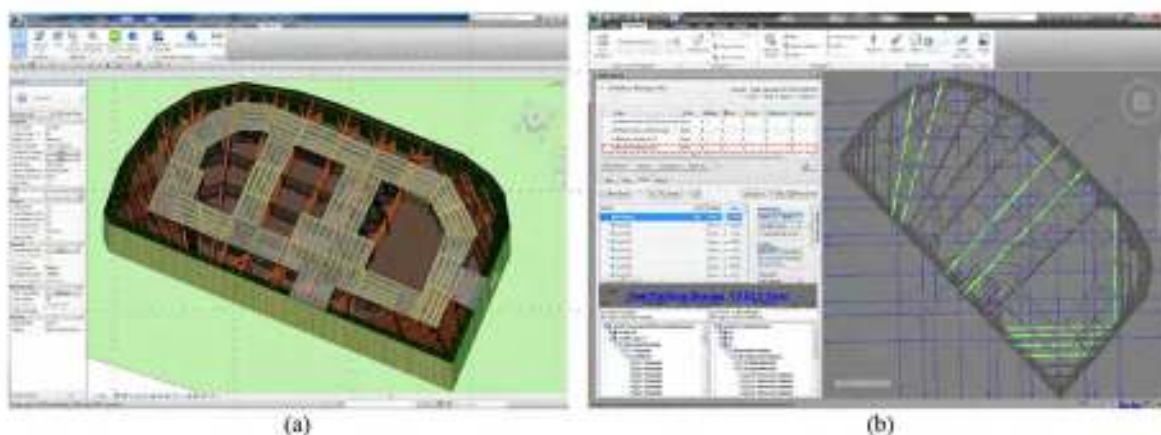


Figure 8: Example of BIM Export to Navisworks for Clash Detection
(a) ELS and Steel Platform on BIM Model, (b) Clash Detection Result by Navisworks

2.5 Design Analysis

BIM data can be exchanged between Revit and different design computer programmes. The foundation/structure elements can be transferred to structural design programme by SAFE/ Oasys GSA analysis and calculation for optimizing the design scheme. The links between Revit and SAFE/ GSA are provided by the Revit plug-in, CSiRevit / GsRevit (see Figure 9 and 10). The foundation / structure in BIM model can be exported for creation of SAFE/ GSA model or synchronizing an existing SAFE/ GSA model with changes from Revit model.



Figure 9: Example of BIM Data Exchange between Revit and SAFE by CSiRevit
(a) Export foundation structure from Revit to create a new SAFE model, (b) SAFE model view

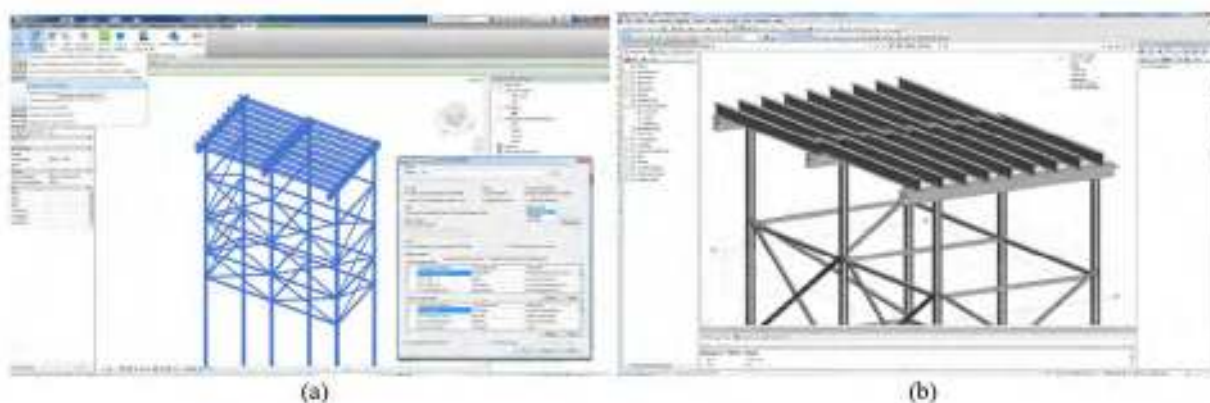


Figure 10: Example of BIM Data Exchange between Revit and GSA by GiRevit
(a) Export steel platform from Revit to create a new GSA model, (b) GSA model view

2.6 Pre-fabrication Design on BIM

BIM can facilitate designing pre-fabricated components (see Figure 11) and applying Design for Manufacture and Assembly (DfMA). All components can be scheduled for quantity take-off, component arrangement planning and connection planning in the BIM environment. The planning is essential for adopting pre-fabricated units, which allows correct manufacturing and speeds up the installation process on site.



Figure 11: Example of Pre-fabrication Component for ELS Construction

2.7 BIM-based Quantity Take-off (QTO)

Traditionally, quantity take-offs and estimates have been performed and developed with 2D drawings supplied by the architect to the contractor. However, in recent years, a shift is observed in the construction industry toward Building Information Modeling (BIM) and BIM-based quantity take-off (QTO), see Figure 12. Although BIM has gained popularity in recent years, models supplied to contractors often lack consistent quality that would allow the contractor to extract data facilitating an appropriate quantity take-off and estimate. The contractor could create its own models.

Family and Type	Work	Length	Unit Weight (kg/m)	Steel Turnage (Tons)
UC-Universal Column: UC305x406x393				
UC-Universal Column: UC305x406x393	001	17932	0.383	6.88
UC-Universal Column: UC305x406x393	001	32164	0.383	12.44
UC-Universal Column: UC305x406x393	001	12784	0.383	5.00
UC-Universal Column: UC305x406x393	001	8482	0.383	3.33
UC-Universal Column: UC305x406x393	001	14420	0.383	5.67
UC-Universal Column: UC305x406x393	001	12362	0.383	4.83
UC-Universal Column: UC305x406x393	001	5815	0.383	2.23
UC-Universal Column: UC305x406x393	001	18987	0.383	7.36
UC-Universal Column: UC305x406x393	001	18140	0.383	7.07
UC-Universal Column: UC305x406x393	001	284	0.383	0.15
UC-Universal Column: UC305x406x393	001	30481	0.383	10.06
UC-Universal Column: UC305x406x393	001	32784	0.383	13.28
UC-Universal Column: UC305x406x393	001	7818	0.383	2.99
UC-Universal Column: UC305x406x393	001	17932	0.383	6.88

Figure 12: Example of BIM Schedule for Quantity Take-off

Through traditional methods, quantification of materials can consume 50 - 80% of the estimating time. Figure 13 illustrates the potential for time saving using BIM for quantity take-off.

Category	Contents	Time (minutes)	
		Manual*	Revit
Foundations	Piling, pile caps, footings, basement wall, beams, slabs	60 - 120	25
ELS	Struts, walings, bracings, spacers, king posts and excavation volume	30 - 80	15
Total		80 - 200	40

Figure 13: QTO Time Saving with BIM

* Time may be various subject to the project scale and complexity

Given the large amounts of time saved and minimal deviations in the take-off, the use of BIM method is advantageous to the project. In addition, Dynamo, a programming tool in BIM, can provide an accurate excavation volume estimation especially when the site is on a hillside (see Figure 14). It provides a quick solution for engineers to compare the design schemes in commercial aspect.

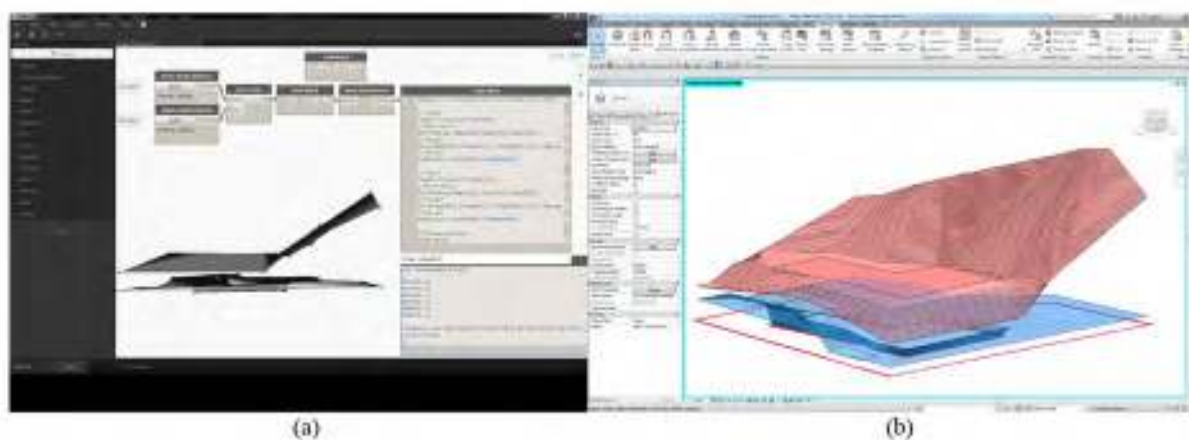


Figure 14: Example of Estimating the Excavation Volume by Dynamo

2.8 Construction Sequence Simulation

BIM can simulate for understanding the progress of construction activities throughout the lifetime of a project. The simulation helps to develop the construction sequence for planning the programme to reduce time delay as well. An example of sequence and phasing of various stages of the construction development is shown in Figure 15.

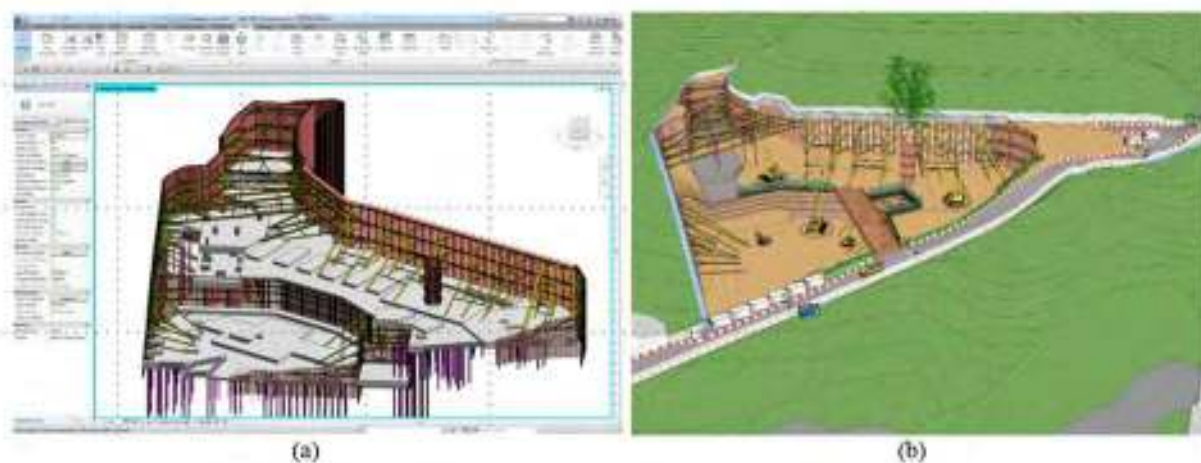


Figure 15: Example of BIM Model to Sketchup in 3D Visualization with Construction Sequence

3 CONCLUSIONS

In conclusion, Building Information Modeling (BIM) and associated digital tools are capable of visualizing the ground profiles with the borehole data, detecting the possible clashes between all structural elements of foundation, ELS members and steel platform; converting the wall/ column schedules in the tender documents into digital formats; transferring the foundation/ structure elements to structural design programme by SAFE/ GSA analysis for selecting an optimum scheme; taking quantities of pile length, concrete/ excavation volume, and steel tonnage in BIM; developing the construction sequence simulation for planning the programme to reduce time delay as well as the cost estimation. The proposed recommendations and best practices as mentioned above are discussed to promote achieving a higher degree of efficiency.

BIM, Big data, cloud computing, and analytics are changing how construction is planned, designed, built and managed. Technology will better connect people, processes, and ideas to create more resilient and sustainable construction industry. The past results and techniques adopted using BIM in tender and construction stages for various foundation and Excavation and Lateral Support (ELS) projects are discussed. The adoption of BIM methodology brings benefits to the foundation projects through technical improvements, reducing the duration of tasks and increasing their efficiency. The new technologies, like 3D printing, robotics, drones, the Internet of Things (IoT) and machine learning, are driving massive changes in how we build things.

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Digital Mapping of Discontinuities

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ABSTRACT

The mapping of discontinuities in rock slopes can be a time consuming and expensive process, with slope access necessitating the provision of double layer scaffolding or the use of specialist rope access techniques and the physical mapping itself taking extended periods as engineering geologists record the key characteristics of the discontinuities present in the slope. Recent advancements in laser scanning and drone photogrammetric techniques mean that it is now possible to overcome these issues and remotely collect highly detailed point cloud data of a rock slope in a matter of hours and at considerably less expense. The availability of this data has opened up a number of new opportunities for the collection and assessment of data on discontinuity characteristics, most specifically the geometric properties including the dip, dip direction, spacing and persistence of surfaces within a rock slope. This paper provides a review of the main data collection and processing techniques as well as the software packages currently available for the analysis and assessment of discontinuities within rock slopes. In this respect, two representative rock slopes were surveyed by techniques including traditional mapping (aided by digital tools), mobile laser scanning and drone photogrammetry. Comparative assessment of the results obtained was then undertaken to determine the effectiveness of the new digital surveying and analysis approaches identified. Recommendations on the best practice for digital mapping and assessment of discontinuities from point cloud data are then provided.

1 INTRODUCTION

The collection and analysis of discontinuity data is a fundamental and essential part of any stability assessment for a rock slope. Traditionally, this data has been collected by engineering geologists making use of compass clinometers to obtain readings of the dip and dip direction of discontinuities, supplemented by pro-forma sheets to record information on measurements and observations of other key factors such as discontinuity persistence, aperture, waviness, seepage, and the presence and nature of any infill materials. For large-scale rock slopes, the provision of double layer scaffolding or the use of 'cherry picker' type mobile platforms is typically required to provide safe access to obtain such readings throughout the entire slope area (Figure 1). Otherwise, scanline or window surveys can only be carried out with limited sampling pool within the accessible portions usually along the toe of outcrops, meaning valuable and important data from the areas above is lost.

The predominantly manual nature of these surveys as well as the need for scaffolding access means that the works can often be time consuming and expensive. Moreover, the intrinsic nature of manual measurement may give rise to problems associated with selective sampling and human bias, especially in the case of inexperienced practitioners.



Figure 1: Examples of Traditional Approach to Discontinuity Surveys

2 RECENT DIGITAL DEVELOPMENTS

A number of digital developments have taken place in recent years that provide new tools and approaches for engineering geologists to collect, store and analyse data on the condition of rock slopes. These include a multitude of mobile phone / tablet-based apps for the collection and storage of discontinuity data, essentially replacing the role played by the compass clinometer, as well as photogrammetric and laser scanning tools and software packages that facilitate three-dimensional surveying of an entire slope face and the subsequent extraction of discontinuity related data from the resultant point clouds generated.

2.1 Smartphone / Tablet-based Apps

One of the most practical and useful digital developments for discontinuity mapping has been the advent of 'apps' that make use of the accelerometers within smartphones and tablets to record the dip and dip direction data of a surface (Figure 2a). In this respect, several apps are commercially available for download for most smartphone / tablet devices and operating systems, with some of the most common being the GeoID app developed by Seoul National University, the Lambert Clinometer developed by the University of Kiel and the Fieldmove Clino app developed by Midland Valley, all of which have similar functionality.

Several papers providing comprehensive reviews of these apps (and many others) together with associated devices and operating systems have been authored, most notably those by Almandine et al. (2017) and Nováková & Pavlis (2019). These reviews found that most of the apps and platforms

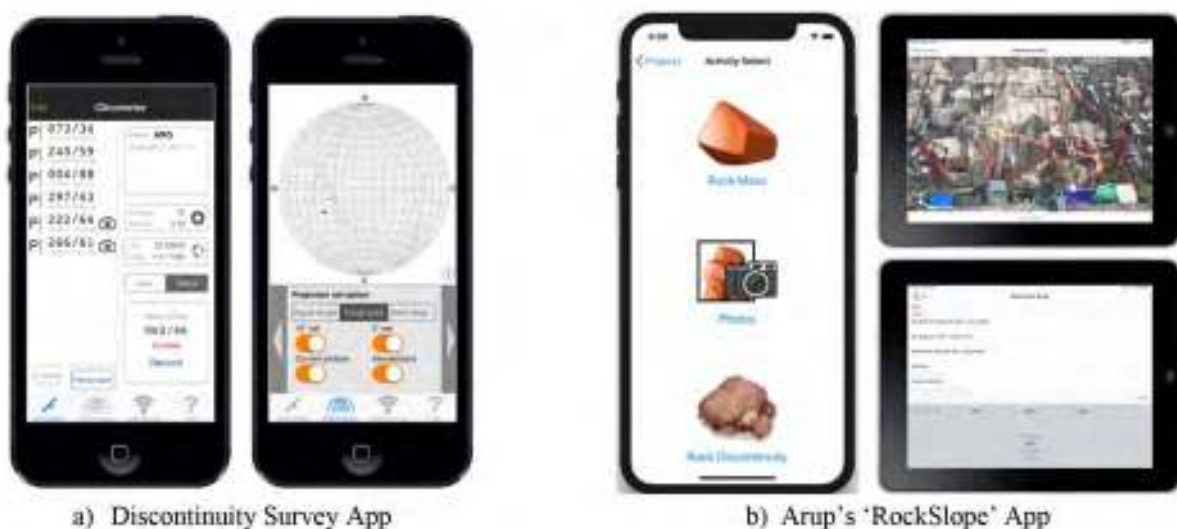


Figure 2: Examples of Mobile Phone / Tablet-based Discontinuity Survey Tools

functioned adequately provided that care was taken to avoidance of metal and electronic objects that could interfere with the magnetic field of the recording device. Given such constraints, it is recommended that an appropriate number of parallel readings always be taken with a traditional compass clinometer to check the accuracy of readings obtained via the chosen app / device against confirmed data.

Whilst a number of apps have been developed to collect and store dip and dip direction data together with photos and notes, few platforms have been developed to comprehensively collect data for an entire rock slope in the manner laid out in the Geotechnical Manual for Slopes (GEO, 1984), namely the integrated collection of full discontinuity details (e.g. inclusive of data on persistence, aperture, waviness, etc.) together with the recording of discontinuity locations and extents on photographic developed elevations of the rock slope.

To address this and facilitate more comprehensive use of digital tools, the authors made use of Arup's in-house research and development fund 'Invest in Arup' to develop a RockSlope app specifically for this function (Figure 2b). This app allows users to photograph a rock face, record details of the rock mass as per Figure 2.10 of the Geotechnical Manual for Slopes (GEO, 1984), directly annotate the photograph with discontinuity locations, each time collecting the data in the typical data sheet for discontinuity survey as per Figure 2.11 of the Geotechnical Manual for Slopes (GEO, 1984). The collected data is then transmitted wirelessly to an in-house Spatial Data Infrastructure (SDI) database, from which mapping reports can largely be generated automatically.

2.2 Point Cloud Data Collection and Processing Tools

Whilst the aforementioned tools provide significant benefits to the ease with which rock slope mapping data can be collected and reported, they do not provide any direct benefits in terms of reducing the cost and effort needed to physically access the slope face for the mapping. In this respect, the Invest in Arup fund was again used to review and assess alternative ways in which rock slopes could be mapped, focusing on the collection and interrogation of point cloud data from remote sensing techniques, specifically photogrammetry of images collected by Unmanned Aerial Vehicles (UAVs) as well as data from Mobile Laser Scanning (MLS). Both of these methods provide a mechanism for collecting a large quantity of three-dimensional survey point data in a non-selective and effective manner. Additionally, the high mobility of the devices allows the slope to be viewed from multiple angles of incidence, essentially overcoming the problem of occlusion that is often associated with fixed terrestrial laser scanning (TLS) surveys (Figure 3).

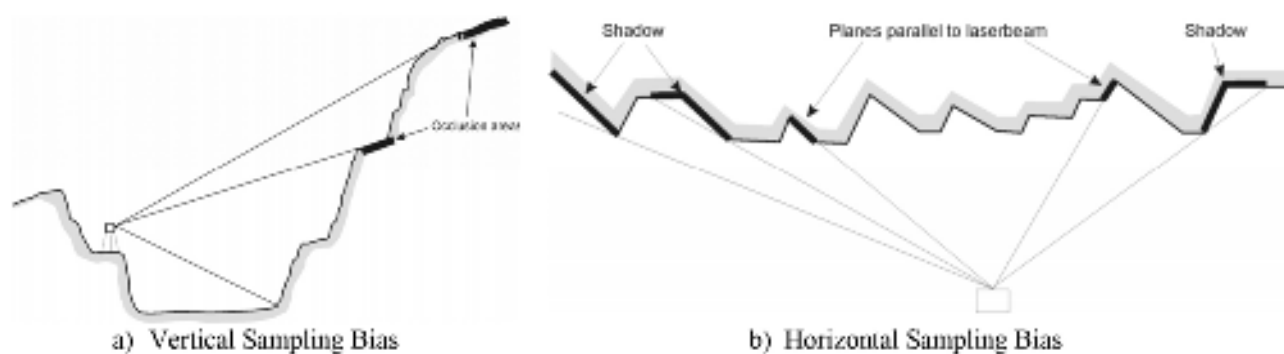


Figure 3: Demonstration of the Problem of Occlusion Associated with TLS (Slob, 2010)

On this occasion, the digital surveys were conducted using a GeoSLAM Zeb Revo RT MLS and a DJI Mavic Pro UAV (Figure 4). The data collected from the surveys was processed to generate point clouds using the GeoSLAM Hub and Agisoft PhotoScan software packages respectively.

The processed point clouds comprise digital outcrop models (DOMs) which, when georeferenced, provide virtual three-dimensional representations of the actual geological outcrops and surfaces (Figure 5). The DOMs are then further processed in specialised point cloud programs such as CloudCompare and analysed with the aid of different algorithms. The increased accessibility and functionality means that the implementation of point cloud technology for geotechnical purposes is



Figure 4: Examples of Survey Tools for the Generation of Point Cloud Datasets

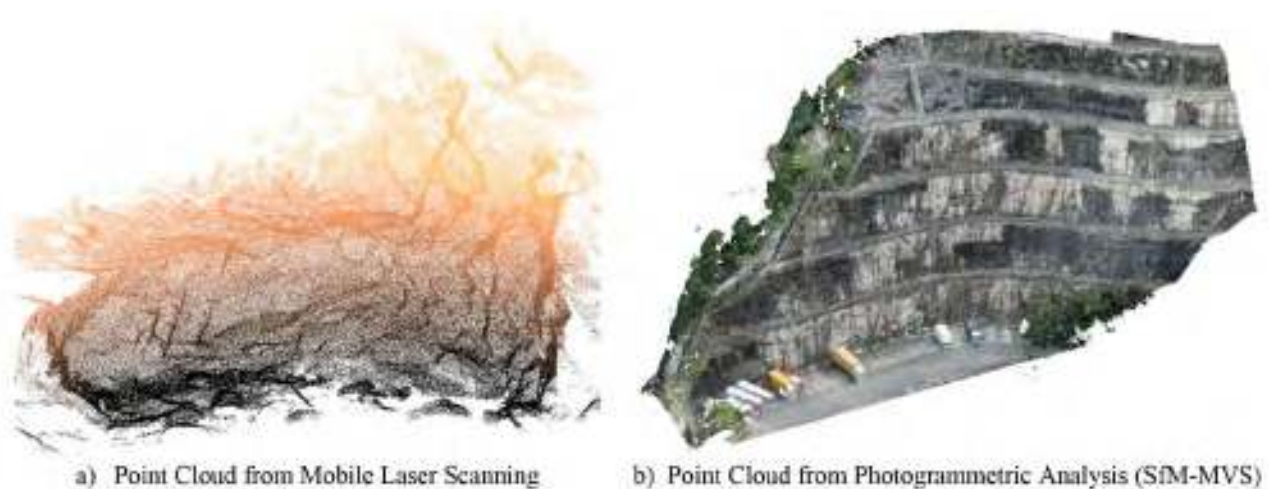


Figure 5: Examples of Processed Point Cloud Data

becoming more commonplace, especially for dimensional measurement, rock mass characterisation, slope monitoring, rock fall detection, mass displacement and change detection (Kemeny & Turner, 2008).

2.3 Point Cloud Assessment Tools

An online search and literature review identified several commercial and open-source tools that can be used for the interrogation of point cloud datasets to extract rock discontinuity data. These included, amongst others, the commercial software packages by Slob et al. (2005) and Jaboyedoff et al. (2007) as well as Matlab-based algorithms by Gigli & Casgli (2011) and Voge et al. (2013). Most of these packages operate in a similar manner and utilise one of the following data processing approaches: i) using least square method to subset the points within the point cloud; ii) calculating normal vectors to a series of 2.5D interpolated surfaces simplified from the 3D dataset; iii) calculating the orientation of each node in interpolated triangular irregular networks (TIN); or iv) calculating the planar orientation from volumetric pixels (Riquelme et al., 2014). However, these approaches can often result in the undesired loss in detail and accuracy.

Amongst the various tools identified and trialled, the FACETS tool in CloudCompare (Dewez et al., 2016) and Discontinuity Set Extractor (DSE) in Matlab (Riquelme et al., 2014) are widely recognised as the preferred packages for automated discontinuity identification due to their high user-friendliness and ease of application as well as the high degree of accuracy in the results generated. The Compass tool in CloudCompare (Thiele et al., 2017) has also gained a degree of popularity for those preferring to manually define discontinuity surfaces within the point cloud as opposed to using semi-automated identification and mapping functions.

3 TRIAL APPLICATION

To facilitate a detailed review of the functionality and benefits of using point cloud data for the mapping of discontinuities within rock faces, comparative surveys and assessments were undertaken for a variety of rock slope locations and conditions in Hong Kong. These included the generation of point cloud datasets from both MLS and photogrammetric analysis of UAV photographs, as well as traditional mapping of discontinuities within the accessible portions of the slopes. Such approaches were selected to facilitate direct comparison of the outputs obtained and validate the effectiveness of the point cloud data analysis packages.

3.1 Site Selection

A total of two study sites were selected for the trial application (Figure 6). These include a long-abandoned quarry slope and natural coastal slope at Lei Yue Mun and an engineered rock slope at Yiu Hing Road in Shau Kei Wan. The section of slope surveyed at the Lei Yue Mun site comprises a small coastal exposure with minor anthropogenic modification. The 6 m-high outcrop at this location is accessible at both the slope toe and crest and can be subdivided into two portions, namely the northern and southern portions dipping at $45^\circ/190^\circ$ and $45^\circ/125^\circ$ respectively. The Yiu Hing Road site comprises a large rock cut slope (Feature No. 11SE-B/C532) that is approximately 97 m high, stands at a slope angle of 65° and dips with an orientation of 330° . Due to accessibility and safety concerns, surveys were only carried out on the lowest bench of the slope from the platform at the slope toe.



Figure 6: Locations of Trial Sites and Survey Areas for Comparative Assessment

3.2 Data Collection

Data for the two trial sites were collected by three different means, namely: i) traditional rock slope mapping at the toe of the outcrop using a digital compass clinometer and the Arup RockSlope app; ii) mobile laser scanning from safely accessible areas at the toe and, where possible, crest of the rock slope; and iii) collection of vertical and oblique aerial photographs from UAV (Figure 7).



Figure 7: Data Collection from the Lei Yue Mun Ex-quarry Site

3.3 Point Cloud Generation and Processing

The generation of point clouds from the raw data obtained by the MLS and photogrammetric surveys required processing in the respective commercial software packages for each device, as summarised below.

Data from the GeoSLAM laser scanner was processed using the GeoSLAM Hub software to generate both .ply and .las files containing the point cloud data. The processing settings adopted 100% of the points surveyed, no spatial decimation and a convergence threshold of two in order to provide relatively rapid yet accurate data. Once processed, the point cloud data files were imported and georeferenced in the CloudCompare software by rectification against existing survey data and/or orthorectified photographs.

The images from the UAV surveys were processed photogrammetrically using the Agisoft PhotoScan software. This is a widely used commercial software package that semi-automatises the image processing procedures using Structure from Motion and Multiview Stereo (SfM-MVS) technique for the automatic calculation of camera positions and scene geometry, as well as mesh and DOM construction. A number of authors have already demonstrated the capability of the SfM-MVS technique for constructing reliable and high-resolution DOMs up to decimetre and centimetre level accuracy, including Micheletti et al. (2014) and Lucieer et al. (2014). Once processed, the data was exported as a georeferenced (HK Grid 1980) .ply file.

3.4 Data Analysis

The processed point clouds were then imported to and analysed on different platforms, primarily using the FACETS function in CloudCompare and the DSE algorithm in Matlab. The data collected from traditional discontinuity surveys was exported from the Arup's RockSlope app for use in the Rocscience DIPS package.

3.4.1 FACETS

FACETS is a CloudCompare plugin that can be used to partially automate the identification and surveying of rock discontinuities from DOMs inside a convenient software environment (Dewez et al., 2016). The package uses a segmentation tool, either Kd-Tree or Fast Marching, to divide the original point cloud data into sub-cells of flat polygons based on their planarity and roughness (Dewez et al., 2016). Subsequently, three levels of clustering are undertaken, namely i) computation of elementary facets; ii) grouping of element facets into planes; and iii) the identification of parallel planes as discontinuity sets (Dewez et al., 2016). The results can be visualised in CloudCompare and exported as 3D shape file and/or comma-separated-variable files (e.g. CSV and ASCII) for further processing.

3.4.2 DSE

Discontinuity Set Extractor (DSE) was developed by Riquelme et al. (2014) and is an open-source

Matlab-based algorithm for identifying, extracting and analysing rock discontinuity from DOMs in a semi-automatic approach. This Matlab tool identifies and defines the algebraic equations of different planes within a rock face by applying an analysis based on a neighbouring points coplanarity test, finding principal orientations by Kernel Density Estimation and identifying clusters by the Density-Based Scan Algorithm with Noise (Riquelme et al., 2014). DSE claims an enhanced accuracy over other packages due to the fact that all of the 3D information stored in every point in the point cloud data is utilised to compute the normal vectors, rather than basing the analysis on simplified surface models as is done by most other packages. Further details on how this is achieved are provided in Riquelme et al. (2014). Another major difference between DSE and other packages is that, for better or worse, DSE focuses on the identification and mapping of statistically significant discontinuity sets as opposed to recording each and every discontinuity surface.

The outputs of both tools can be visualised within the respective software package itself, in terms of both dip and dip direction on stereoplots as well as colour-coded surface plots within the point cloud model itself (Figure 8). The FACETS tool also allows the data to be exported as .csv or .shp files for subsequent use in structural geology packages such as DIPS or ArcGIS. In addition to dip and dip direction,

Lei Yue Mun

Yiu Hing Road

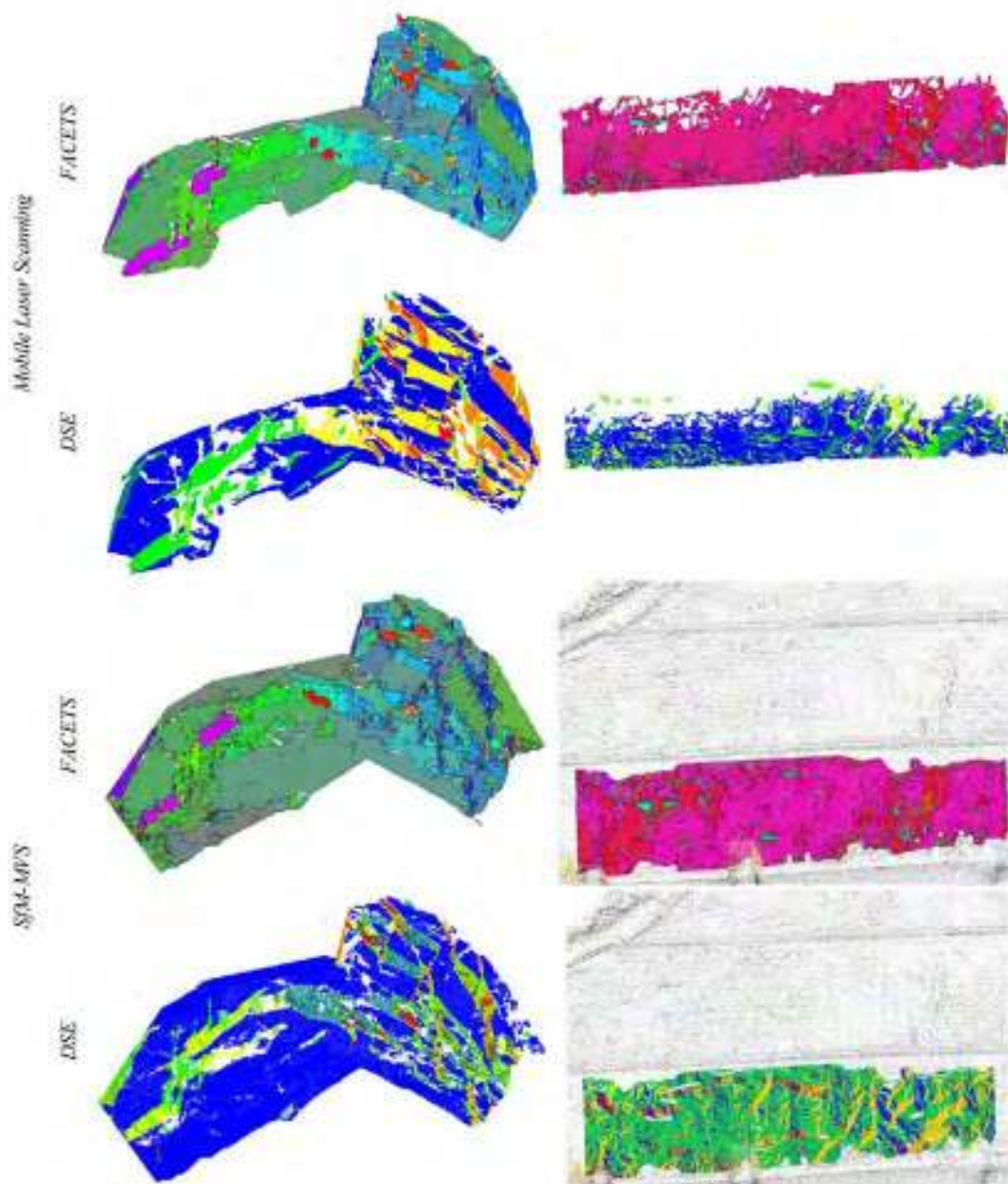


Figure 8: Examples of Discontinuity Surfaces Identified from Point Cloud Datasets

the exported .csv also includes data on the location of the surface (Easting and Northing of centroid), the horizontal and vertical extent of the discontinuity as well as its surface area. The availability of this data essentially means that the mapping data can be filtered not only by dip and dip direction, but also in terms of the apparent discontinuity persistence.

3.5 Comparative Assessment of Different Approaches

3.5.1. Data Acquisition, Processing and Cost

Both of the remote sensing techniques adopted for this trial provided significant benefits and advantages in terms of their ability to collect comprehensive data throughout the slope face, whilst at the same time overcoming the issues of safety and physical accessibility that are commonly associated with traditional rock slope mapping. However, whilst providing significant benefits in most circumstances there are some limitations to the use of the remote sensing techniques in terms of collecting clean point cloud data, in particular for slopes with wire mesh cover or slope vegetation. Due consideration of the aforementioned problems associated with angles of incidence from the instruments should also be made. Additionally, the mobile laser scanner used in this trial has a relatively limited range of about 30 m. This results in a diminishing density of point cloud data away from the scanner location, with areas further than 10 to 15 m from the scanner typically yielding insufficient data for reliable analysis. In comparison, the range of SfM-MVS is theoretically limitless in terms of the distance of survey but has declining accuracy for the models generated as distance from photograph source increases. In this respect, the lower the flight height and the greater the degree of overlap between photographs, the more reliable the resultant point cloud model will be.

Overall, the adoption of MLS and SfM-MVS technology was found to significantly reduce the time required to survey the sites, with less than 1 hour (excluding post-processing) needed to map the whole area in contrast to the traditional survey, which takes about 1 hour for just 10 m² of mapping surface. In addition, the post-processing times for point cloud generation were typically less than 1 hour for the MLS but upwards of 8 hours for the generation of high resolution point clouds using SfM-MVS. Despite the seemingly long time required for SfM-MVS, most of this time is user-free, with the software simply left running in the background or overnight. As such, the overall effort required from the user is still minimal.

The data from the SfM-MVS has the benefit of being georeferenced with x-, y- and z-coordinates for whatever coordinate system is selected during the data processing stage, whereas the MLS-derived point cloud requires the user to georeference the processed point cloud against a separate pre-existing survey dataset. This additional step may induce an extra level of systematic error for sites where the laser scanning does not include properly surveyed ground control points.

Despite these obvious advantages, it should be noted that the capital outlay for MLS and SfM-MVS is considerably higher than the cost of a simple compass clinometer, with the full package (including processing software) for the MLS (GeoSLAM scanner and GeoSLAM Hub software) and SfM-MVS (UAV and Agisoft PhotoScan/Metashape software) costing approximately HKD\$450,000 and HKD\$40,000 respectively.

3.5.2. Result Accuracy and Precision

The results of the comparative assessment (Figure 9) indicate that the accuracy and precision of the three surveying approaches adopted are broadly consistent in terms of the dip and dip direction readings for major discontinuity sets, although variance of up to about 10° to 15° is noted in terms of the centroid of the discontinuity set due to the number and scatter of individual readings recorded within the set. In this respect, it was found that the stereoplots from traditional survey and MLS typically agreed better with one another than those generated from SfM-MVS. The minor discrepancy between the outputs of these approaches and SfM-MVS most likely arises due to a measure of image distortion from the photogrammetric processing. In this regard, Eltner et al. (2016) suggest that SfM-MVS can only produce point clouds of accuracy and completeness comparable to laser scanning under ideal conditions and that the quality of the point clouds generated can be affected by factors such as the complexity of

the slope geometry, lighting and weather conditions, the quality of the camera used, and the stability of the drone used for the survey, etc.

As is evident in Figure 9, notable variance in the densities of discontinuity pole readings is apparent between the various data collection and processing techniques. This is largely a function of the coverage of the acquired dataset, with incomplete and selective sampling occurring during traditional surveys as compared to the remote sensing due to the inaccessibility and unconscious human bias issues associated with this approach, and seemingly less data output from the DSE modelling approach due to its focus on discontinuity sets rather than an entire dataset. Additionally, although seemingly providing more thorough assessments the quality of the remote sensing data is highly affected by the density and resolution of the point cloud generated, in particular the criteria selected for interpolation and dense cloud construction in the SfM-MVS workflow.

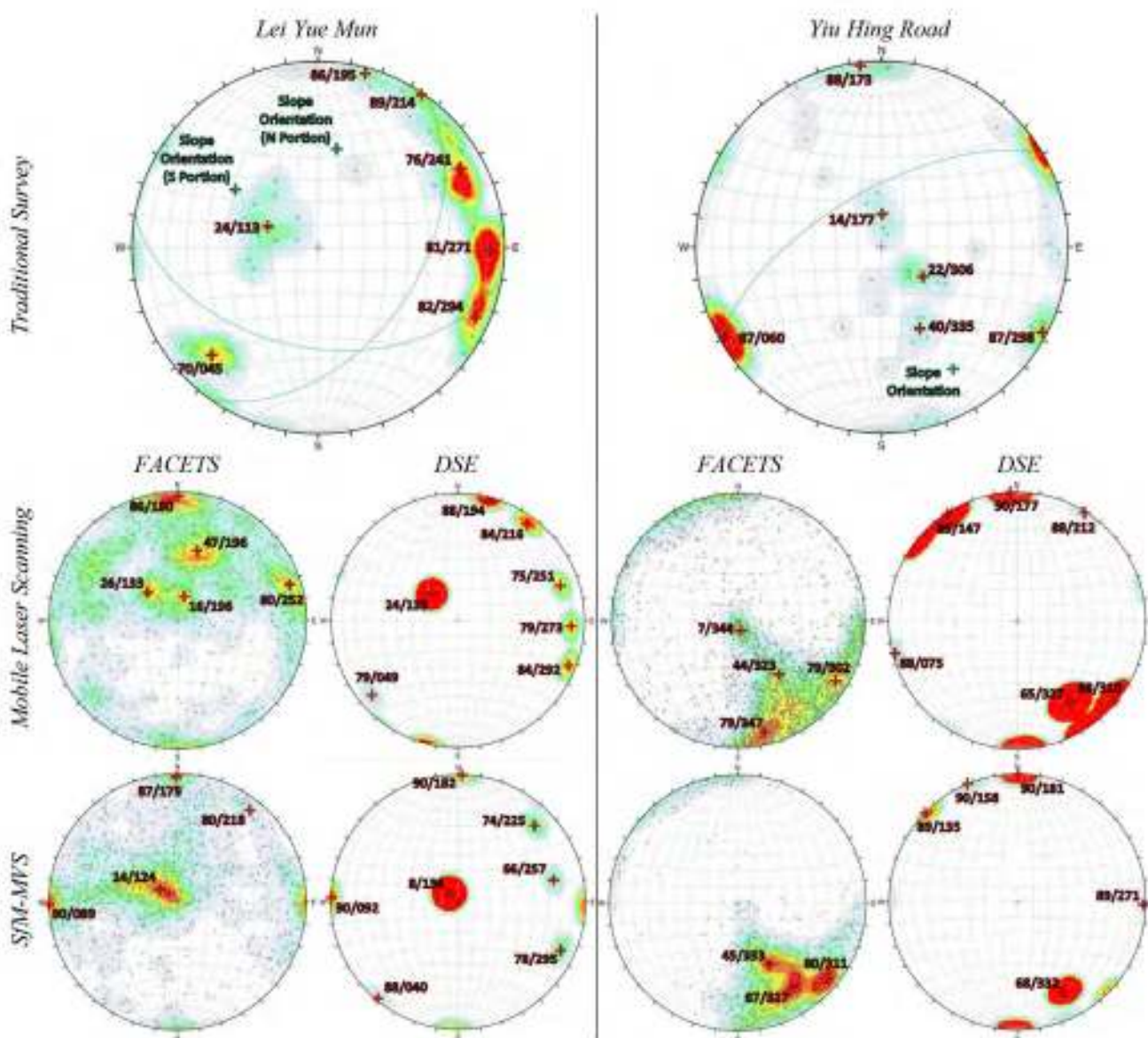


Figure 9: Stereoplots of Data from Various Sources

Overall, the quality of the dataset from the remote sensing techniques can be directly correlated to the number of survey points on the surfaces in the point cloud used to calculate planarity. For engineered slopes with relatively uniform profiles, as in the case of Yiu Hing Road, the orientation of the cut slope face can predominate the natural discontinuity data extracted from the point cloud due to its extensive presence, resulting in masking of some of the smaller more subtle features present in the

slope. This can also result in an insensitivity to any smaller-scale discontinuities within the rock face. A good example of this bias can be found in the data for the Yiu Hing Road rock slope (Figure 9), where the data from both SfM-MVS and DSE shows clear favour towards surfaces striking sub-parallel to the rock face (i.e. engineered surfaces). Most notably, this bias appears to have masked the presence of the main discontinuity set ($87^{\circ}/060^{\circ}$) recorded from the traditional mapping, further suggesting that data for sub-vertical discontinuity sets striking perpendicular to the slope forming face may not always be well expressed as measurable surfaces in point cloud data.

The data from the remote sensing analysis also shows a tendency to average readings along a surface to provide a single orientation reading. Although useful for general reviews of kinematic stability, this approach means limited consideration may be given to beneficial effects that large-scale discontinuity roughness has on rock slope stability, where present.

3.5.3 User-friendliness, Computation and Visualisation

FACETS is a very user-friendly tool which has been seamlessly implanted within the CloudCompare software package and provides nearly instant visualisation of the results. The stereoplots generated by FACETS in CloudCompare also include interactive functions that facilitate the selection of localised discontinuity clusters, with corresponding updates to the surfaces shown in the point cloud model. Such functionality is extremely useful for reviewing the specific locations of adverse surfaces (or combinations thereof) that may require stabilisation. However, the resultant cluster planes picked up by FACETS do not accurately retain the original shape of the discontinuities and users may thus suffer difficulties in correlating the resultant cluster planes to the actual discontinuities (Figure 10a). As such, this function should only be used to provide information on the approximate positions of the discontinuities of interest.

For DSE, a longer computation time is required and the outputs cannot be viewed directly. Even so, the software package allows users to view preliminary and intermediate results for the refinement on the input parameters. Without the process of dividing the rock outcrop into flat polygons like FACETS, the original shapes of the discontinuities can be preserved. The colour-coded point clouds provide excellent visualisation and facilitate users to spot out different discontinuities easily (Figure 10b).

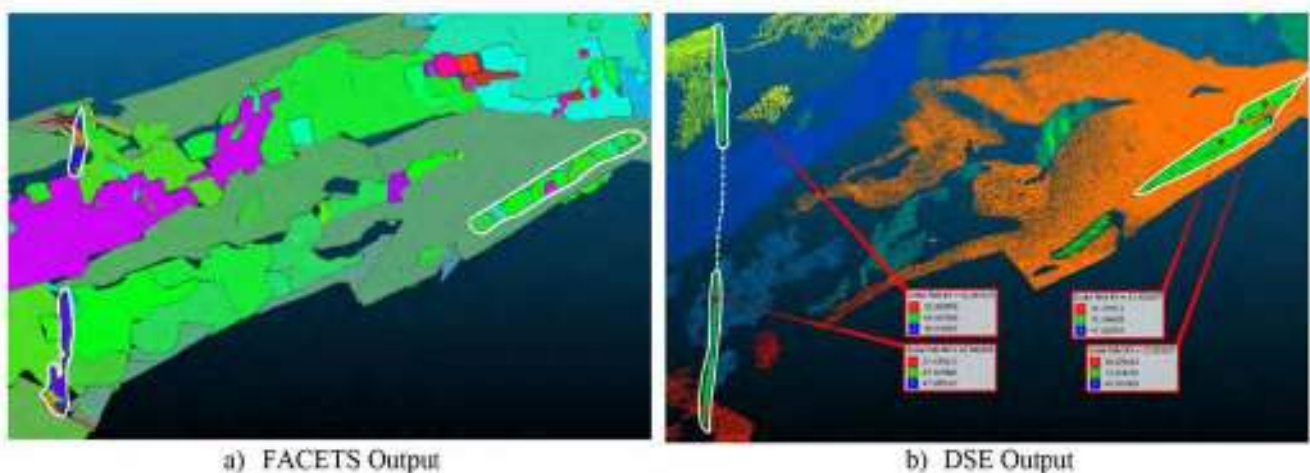


Figure 10: Comparison of FACETS and DSE Datasets

3.5.4 Comparison of Point Cloud Analysis Tools

The results of the trials conducted indicate a high degree of consistency in the data obtained from the two analytical approaches, but with clear benefits and drawbacks to each of them. As previously mentioned, FACETS calculates the dip and dip direction value for each and every identified surface within the point cloud and generates stereoplots and data files for all of these measurements, with the .csv export files facilitating subsequent review and filtering of this data for subsequent analysis in other packages such as Rocscience DIPs. The high density of data obtained from FACETS provides a comprehensive record of discontinuity conditions. However, without filtering for persistence, the high density of data can result

in diffusion of the discontinuity set data and drift in the estimated set orientation as relatively minor discontinuities are afforded the same statistical significance as highly persistent surfaces.

Unlike FACETS, DSE automatically identifies the major discontinuity sets through the application of density functions, which can help reduce the impact of unconscious bias in manual discontinuity set determination. Although this approach means that the dip and dip direction data for specific discontinuities and discontinuities with low statistical significance cannot be directly retrieved from DSE, DSE does perform better clustering or grouping of identified planes, thanks to the utilisation of the true 3D information stored in the point cloud dataset. This means that DSE is capable of identifying discontinuities lying along the same structural trace, but which have become disconnected due to the cross-cutting relationships among them (Figure 10b). Such approaches prevent the overestimation on the population of discontinuities forming a set and the issue of set data drift identified within FACETS datasets.

The computed normal vector information from DSE can also be further processed to calculate the persistence and spacing of discontinuity sets according to the approaches stated in Riquelme et al. (2018) (Figure 11). Although the accuracy of the persistence and spacing needs to be further verified with extra case studies, DSE has the potential to be further developed to extrapolate data for rock mass parameters such as RQD, Q-value, RMR, etc. For FACETS, the potential for such further development may be obstructed by the questionable quality of clustering and grouping analysis.

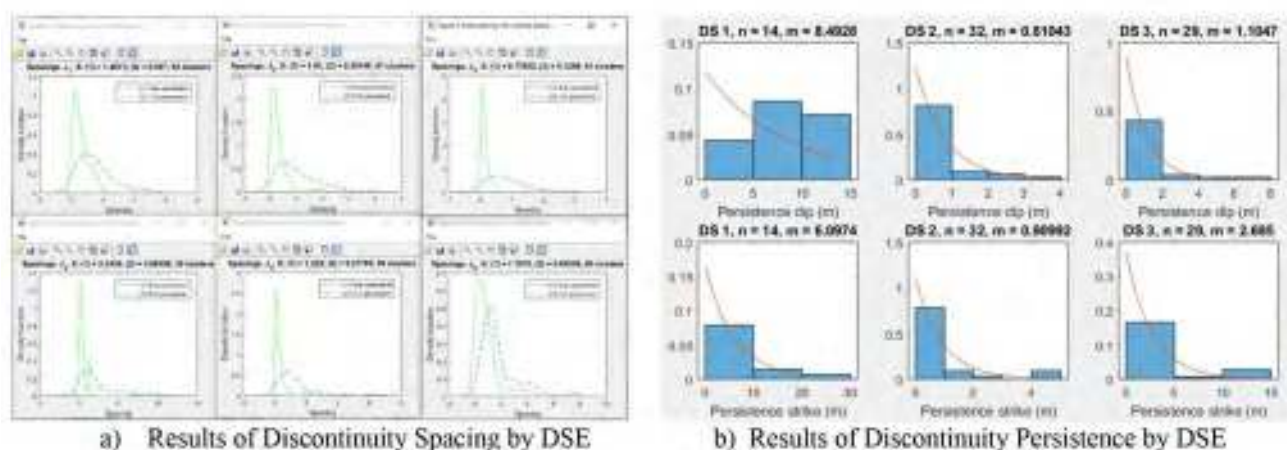


Figure 11: Computation of Discontinuity Spacing and Persistence by DSE

4 CONCLUSIONS & RECOMMENDATIONS

The emergence of new digital and remote sensing techniques in recent years has revolutionised the way in which rock slopes can be mapped and analysed. The accelerometers and apps within our smartphones and tablets now justify that traditional mapping can be conducted with much higher efficiency and the availability of remote sensing tools such as MLS and photogrammetry mean that slopes can now be mapped in their entirety in a matter of hours rather than days. The point cloud data obtained from the remote sensing also provides the engineering geologists with a mechanism to further review and interrogate the mapping data after the initial survey has been completed, something that would require repeated site visits following a more traditional approach.

Although MLS and photogrammetric surveys can both yield high quality point cloud datasets, in terms of data quality the models derived from MLS were found to provide higher resolution and more reliable datasets than those obtained by SfM-MVS. However, the quality and resolution of the data from MLS were found to be highly dependent on the proximity and orientation of the scanner with respect to the rock slope, with the best results obtained for rock faces less than 10 m in height where access could be gained to both the top and bottom of the slope. The MLS produced notably lower quality models for rock faces with bench heights exceeding 10 m or rock faces with no access to the slope crest. In such cases, the use of UAVs yielded better quality and more complete point cloud datasets. For the best result, the adoption of UAV-mounted laser unit may offer an all-around solution which retains the advantages of both MLS and photogrammetric surveys.

The rapid and easy data collection from remote sensing has been complemented by the development of several user-friendly freeware tools developed specifically to extract discontinuity data from point cloud datasets, in particular the FACETS and DSE tools reviewed herein. Whilst notable differences exist in the way these tools interrogate the point clouds, with FACETS providing a high density of data for all discontinuities and DSE providing interpretive discontinuity set data, analysis and review of data for several rock slopes indicates that both tools provide reliable mechanisms for determining the structural geological regime within a rock slope. Indeed, the differing approaches and datasets can even be considered complimentary to one another in terms of providing a more well-rounded and holistic assessment for a rock slope.

Although providing numerous and obvious benefits in terms of data collection and analysis, these remote sensing techniques and analyses still require a degree of manual handling and expertise, in particular with regard to sites requiring the removal of vegetation (either from the site itself or from the point cloud model), the georeferencing of the point clouds from MLS, and the selection of appropriate user-defined parameters for both point cloud generation as well as subsequent discontinuity evaluation. The results obtained from point clouds are also the product of a purely geostatistical analysis of rock face geometry. Whilst the resultant datasets provide good factual representations of the outcrop condition and its geometric properties, they include no data on the nature and condition of the surfaces present (e.g. discontinuity roughness, aperture, seepage, and the presence and nature of any infill) and may even oversimplify the geometric condition of the surface through averaging out of plane orientations and the removal of aspects such as roughness / waviness.

As such these approaches and tools should only ever be treated as complimentary to traditional rock slope inspection and mapping by an experienced engineering geologist. To ensure reliability, the data extracted from the point clouds should also be reviewed, checked and calibrated against a reasonable amount of data obtained from traditional mapping approaches.

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Innovations In Wireless Monitoring

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ABSTRACT

Comprehensive monitoring data delivered in a timely and transparent manner to stakeholders is an essential part of risk management and engineering control in construction. This lesson, learnt from previous disasters, has recently been reinforced by a number of issues facing the local Hong Kong Construction Industry. However, although deployments of the latest wireless monitoring technologies in Hong Kong have demonstrated that they are sufficiently accurate, robust and cost effective to allow the automation of many construction monitoring tasks, there does appear to be a reluctance to specify and use these systems. The authors trace development of geotechnical related wireless monitoring in Hong Kong over the last two decades highlighting the advantages and difficulties of several systems used on projects. Wireless monitoring systems are usually automated but they are not “magic” and it is therefore important to be aware of the practical issues likely to be encountered when using them on construction sites. Innovative monitoring can help optimize design, improve control and minimize risk. The authors believe the case for widespread adoption of wireless monitoring in Hong Kong Construction is compelling and challenge all stakeholders to further improve risk management and engineering control on projects by routinely specifying such systems.

1 INTRODUCTION

The findings of investigations into major geotechnical engineering disasters, such as the 1994 Heathrow tunnel collapse and the 2004 Nicoll Highway collapse, commonly include factors such as inadequate or ineffective instrumentation, monitoring and interpretation regimes which either cannot detect or do not communicate critical adverse trends to the key stakeholders in an appropriate time scale. One of the key lessons from these disasters is that, where instrumentation results are used to manage risks, the monitoring should be continuous, automatically processed and immediately made available to all appropriate members of the Operations and Engineering Management teams to allow early identification of unsatisfactory trends so that corrective actions can be taken.

Successful deployments of the latest wireless monitoring technologies on recent and current projects have demonstrated that they are sufficiently mature, accurate and robust to allow cost effective automation of a wide range of construction monitoring activities to ensure quick and reliable communication of the monitoring data. Wireless monitoring therefore provides a powerful means to ensure the lessons learnt can really be implemented on site. There does, however, still appear to be a general reluctance to specify or to use automated monitoring systems in Hong Kong and the authors hope that, by sharing the problems and successes they have encountered, other stakeholders in the construction industry will be encouraged to adopt more of the new innovative automated monitoring systems becoming available.

2 BRIEF HISTORY

The authors have been involved in trialing a number of different types of wireless monitoring systems, primarily for geotechnical monitoring, in Hong Kong over the last two decades. Descriptions of these follow and the advantages of or issues found with each system are briefly discussed.

2.1 Bespoke wireless system and GSM (1999 to 2000)

In early 1999 Gammon Construction Limited (GCL) was one of three contractors commissioned by the Hong Kong Government's Geotechnical Engineering Office (GEO) to carry out trials of automatic piezometric data acquisition systems and, in particular, GCL was to trial a system using radio waves as a means of data transmission. Details of the trials were publicised in December 2000 as GEO Technical Note No. TN 8/2000 and are currently available as Section 2 of GEO Report 117 (see Wong (2001)).

The GCL trial was carried out from July 1999 to October 2000 on two separate sites along Ching Cheung Road and Po Shan Road respectively. Two vibrating wire piezometer sensors were installed in existing standpipe piezometers and cables run from the sensors to a centralized "site subscriber" unit comprising the power supply, data logging and radio transmission systems on each of the sites. The vibrating wire readings were recorded every 5 minutes by the data loggers and the data retrieved from the site subscriber units at regular intervals via a centralized control centre system. The photographs in Plate 1 illustrate the equipment used and the setup at the Ching Cheung Road site.



Plate 1: Ching Cheung Road site equipment and setup

The setup of the site subscriber unit at the Po Shan Road site and a photograph of the control centre are shown in Plate 2.

The data collected at the control centre were imported into spreadsheets for conversion into m head of water and presentation in the form of water head time series charts.

At the end of the trial it was concluded that the system worked fairly reliably and that the transmission of data using both radio waves and GSM was successful. There were, however, initial problems with sensor stability and intermittent issues with loss of data caused by battery discharge during thunder storms and with vibration or electrical interference causing isolated “spike” readings. The data transmission system was not particularly power efficient and consequently the data could not be collected very frequently even with the relatively large batteries and solar panels.

The estimated cost of setting up and operating this sort of system at the time of the trial was in the order of HK\$270,000 for a site with 4 piezometers over a period of 17 months including two wet seasons.

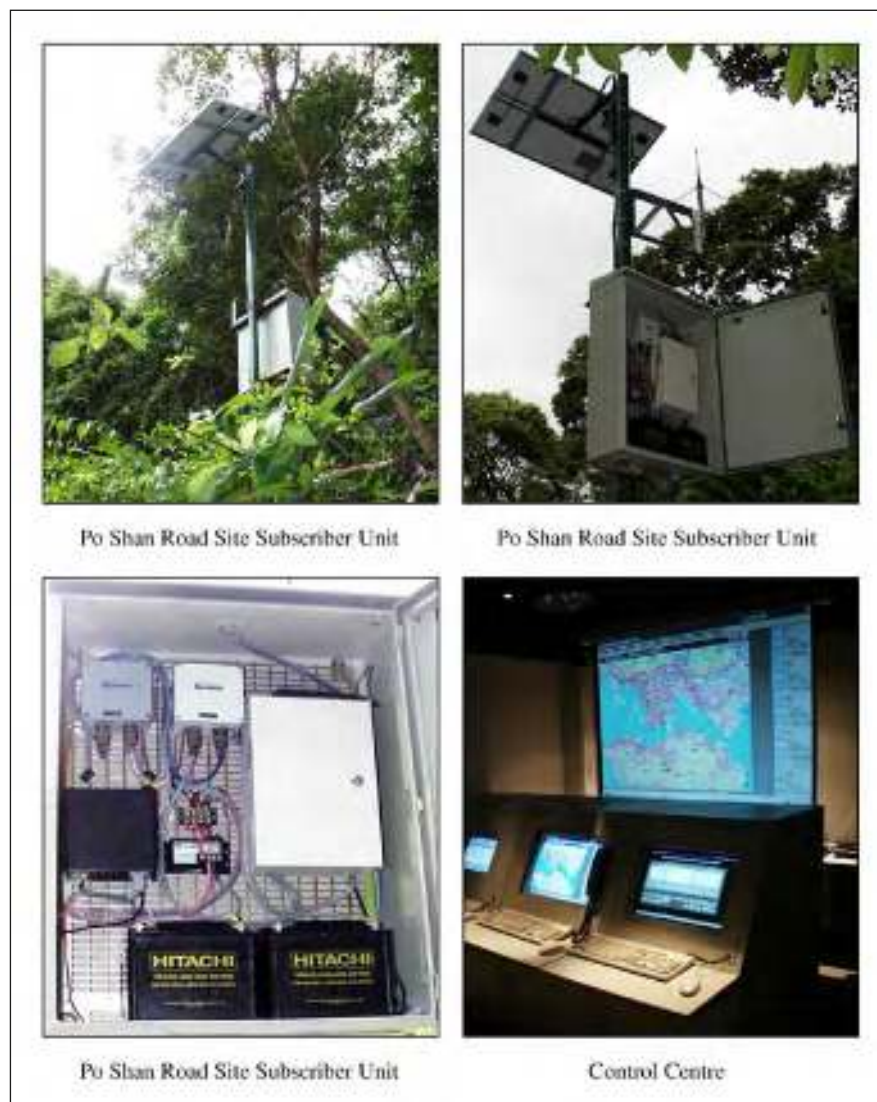


Plate 2: Po Shan Road site subscriber unit and control centre

2.2 Cable free tilt sensors (2005 to 2009)

Soil Instruments Limited (<https://soil.co.uk>) “cable free” tilt sensors were used to monitor movement of structures along Canton Road and Kowloon Park Drive Flyover during excavation and tunneling works carried out from 2005 to 2009. The system comprised 3 radio loggers and 49 cable free electro-level tilt sensors which were pre-set to transmit readings at 15 minute intervals.

The photographs in Plate 3 show the setup of one of the radio loggers and its antenna in the site office.

The other two loggers were installed in waterproof enclosures on other sections of the site further along Canton Road and in Salisbury Road.



Plate 3: Cable free radio logger and antenna setup

Typical installations of the cable free tilt sensors are shown in Plate 4.



Plate 4: Cable free tilt sensors

Each radio logger and the nearby cable free sensors form a star network with data communicated from the sensors to the radio logger. The wireless sensors take the tilt readings at the preset interval and then transmit the values. No readings are stored at the sensor and the sensors do not receive any information back from the radio logger. If a radio logger receives a transmission from one of the sensors it stores the timestamped data in its internal memory. Proprietary software running on a Microsoft Windows PC is used to retrieve the readings stored in the radio logger memory at scheduled times. On this particular project Windows Task Scheduler was used to regularly run a script to automatically upload the data retrieved from the radio logger to the GEOMON system (see Frame (2005)) so that stakeholders could access the latest readings and charts over the Internet.

The system performed really well despite initial concerns about stability of the electro-level sensors and reliability of the wireless transmission system. The main disadvantage with these types of system is that no data are stored on the sensor nodes so if there is a blockage in line of sight during the data

transmission and the radio logger cannot receive the data then it is lost. There is no way for the logger to request a resend or for the sensor nodes to route the data transmission around the blockage. Another risk with proprietary systems is that they are very dependent on continued support from the supplier. In this case, shortly after the completion of the project, Soil Instruments decided to develop a new wireless system based on mesh network technology and stopped supplying the older system primarily because of difficulties in getting continued support for the older version radios from their wireless system supplier.

2.3 Wireless automatic ground water monitoring devices (2010 to 2016)

252 wireless automatic ground water monitoring devices (WAGMD) were installed to automate the monitoring of ground water drawdown at 84 piezometer locations on Hong Kong Island between North Point and Say Ying Pun during deep tunneling works carried out from 2010 to 2016. The WAGMD comprised Insitu Inc (<https://in-situ.com>) LevelTROLL downhole pressure sensors and data logger devices connected to proprietary wireless data transmission units (WDTU) which were supplied, installed and operated by Fugro Geotechnical Services Limited (FGS). The WDTU consisted of a battery power supply and a bespoke interface unit developed by FGS to retrieve readings from up to three LevelTROLL devices and transmit them to a control computer via a GSM/GPRS cellular modem. The complete system was designed to fit into a 250mm x 250mm x 300mm deep recess inside the piezometer surface box as shown in Plate 5.



Plate 5: WAGMD installation in a piezometer surface box

The WAGMD systems performed extremely well, recording and uploading readings every 30 minutes throughout the tunneling works, without any significant problems. All readings retrieved by the FGS control computer were automatically pushed onto the project GEOMON system for exceedance checking and to allow stakeholders to easily access the ground water level data over the Internet.

It was found to be difficult to carry out regular response testing in the piezometers after the WAGMD system had been installed but this was related more to the contract requirement to install the whole system in the surface box rather than the WAGMD system itself. This was worked around by installing short lengths of tubing with appropriate fittings to the tops of the piezometer pipes to allow the piezometers to be filled with water without having to remove the WDTU equipment before carrying out the response testing.

The WDTU batteries did need to be replaced on a regular monthly basis because no above ground installation was allowed which prevented the use of solar panels to recharge the batteries. The system

did not allow remote resetting of the reading intervals which would have been useful during the response testing.

Installation of a WAGMD unit up to 100m deep in an existing piezometer cost around HK\$13,000 and the operating cost was in the order of HK\$50 per sensor per day.

2.4 VWlog8 GPRS (2014 to 2017)

The Soil Instruments Limited VWlog8 was an 8 channel data logger (see Plate 6) which used a Fast Fourier Transform (FFT) algorithm to read vibrating wire instruments and FTP over GPRS to upload the readings to a remote server. It was designed so that a single 12V 7Ah battery would allow 8 vibrating wire instruments to be scanned on an hourly interval and readings uploaded twice a day for a period of 6 months. All readings and the data logger configuration settings were stored on a SD card to ensure no data loss even if the mobile GPRS connection failed. The settings could be changed remotely by updating a copy of the configuration settings on the FTP server and these would be downloaded and applied the next time the logger contacted the server to upload readings.



Plate 6: Soil Instruments Ltd (previously itmsoil) VWLog8

The original version performed well during site trials and, with the cost at under HK\$10,000, it appeared to be a very cost effective way to automate the reading of vibrating wire instruments. One was subsequently used in a tunnel to record readings from initially 2 and later 3 vibrating wire piezometers installed behind the tunnel lining from late 2014 to mid-2017. The data logger worked reasonably well but communication with the FTP server would fail if the server was down requiring a restart of the logger which was a problem if it was not readily accessible. The reliability of the GPRS connection obviously also depended very strongly on the mobile network used.

The supplier did upgrade the firmware in the loggers twice but the reliability issues persisted and, in some cases, the upgraded systems appeared to have more issues than the original version. In addition many corporate IT departments discourage the use of FTP because of security concerns and consequently the authors no longer use any VWlog8 units despite the cost advantage.

2.5 Loadsensing LoRaWAN (2016 to 2019)

Loadsensing's (<https://www.worldsensing.com/product/loadsensing>) LS-G6 LoRaWAN system uses a low power, wide area (LPWA) networking protocol (<https://loro-alliance.org/about-lorawan>) designed to connect battery operated devices to the Internet. Gateways running internet servers are deployed in a "star of stars" topology relaying messages between end-devices and the Internet. Bi-directional communication between the gateways and the sensor devices are supported - typically up to 15km in open areas or 2km in urban areas. The LS-G6 system allows direct access to the internet server running on the gateways so that they can be operated completely independently of Loadsensing. File upload to a remote FTP server is supported and the data on the gateways can also be accessed with direct cable connection using any Android device, via a web HTTP API or by directly down loading CSV or

Microsoft Excel files over the Internet. Loadsensing supply geotechnical and structural monitoring LS-G6 sensors and interfaces such as bi-axial tilt sensors and vibrating wire interfaces in single or multi-channel configurations.

The authors carried out an initial trial of an early LS-G6 system between two construction site offices, about 1.5km apart without line of sight between the gateway and the vibrating wire node, in the West Kowloon area which confirmed the system did work. Subsequent tests carried out in the Ma On Shan area during the period 2017 to 2018 showed firstly that the gateways should preferably be connected to a permanent power supply and secondly that it was not always feasible to get a single gateway installed in a suitable location to reliably cover a site despite the long transmission distances supported. Care also needs to be taken to ensure that gateway antenna installed at height and nodes installed in exposed areas on slopes are protected from damage by lightening during thunderstorms.

A LS-G6 system is currently being used to automate the monitoring of standpipes and piezometers for cut and cover tunnel excavation works in the Kowloon area (see Plate 7) and performance to date has been reasonably good but in some cases nodes have had to be shifted or an additional gateway installed to ensure all the nodes can connect to a gateway.



Plate 7: Ground water level monitoring using Loadsensing LS-G6-VW node

2.6 Wisen Innovation mesh network (2018 to 2019)

Wisen Innovation (<http://www.wisencn.com>) develops and supplies wireless infrastructure monitoring equipment. Their WISENMESHNET® mesh network technology was originally developed as part of work undertaken at Cambridge University (<https://www-smartinfrastucture.eng.cam.ac.uk>).

Wisen supply gateways and a variety of tilt, laser distance, vibrating wire interface, camera and other sensor nodes which are typically installed on sites in groups with distances between neighboring nodes not more than about 100m apart. The nodes support 2 years of monitoring with readings taken at 15 minute intervals without changing or supplementing the battery power supply and the mesh radio network is designed to be resilient re-routing wireless traffic between nodes if needed. Both the nodes and gateway store data locally in the event there is a temporary radio blockage. Data are transmitted from the gateways to Wisen's cloud based visualisation platform using a mobile internet connection. The data may be accessed directly on the visualization platform using a web browser or pulled off automatically by other Internet based systems, such as GEOMON, using HTTP/HTTPS web services.

Wisen's vibrating wire nodes have been used since March 2018 to monitor strut loads for a large braced excavation. A 4-channel node has been installed at each strut location to monitor the 4 vibrating wire strain gauges installed on the steel sections as shown in Plate 8. The data have been collected at intervals ranging from 1 minute, during preloading, to 30 or 60 minutes at all other times and are pulled off the visualization platform by GEOMON which checks for any new data every minute. The

performance of the system to date, in terms of ease of setup, quality of readings and overall reliability of data transmission has been exceptionally good.



Plate 8: Strut load monitoring using a Wisen 4 channel vibrating wire node

3 INDICATIVE COSTING

Indicative costs to install each of the systems described above based on four tilt or piezometer sensors monitored for a period of two years are shown in Table 1. The unit costs in 2019 have been calculated by escalating the approximate costs at the mid-range of the respective deployment periods up to the present day.

Table 1: Approximate system cost comparison based on 4 sensors for 2 years

System	Install Cost (HK\$)	Running Cost (HK\$)	Total Cost (HK\$)	Unit Cost (HK\$/no/day)	Year	Unit Cost in 2019* (HK\$/no/day)
Bespoke Wireless	140,000	187,000	327,000	111.99	2000	198.87
Cable Free Tilt	66,000	304,000	370,000	126.71	2007	206.78
WAGMD	52,000	146,000	198,000	67.81	2013	86.68
VWLog8	40,000	24,000	64,000	21.92	2016	29.44
Loadsensing	49,000	24,000	73,000	25.00	2019	25.00
Wisen Mesh	36,000	30,000	66,000	22.60	2019	22.60

*2019 costs calculated based on CEDD Civil Engineering Works Index in 4th Quarter 2018 and 2nd Quarter in the preceding years.

Whilst these costs are not very precise they do indicate there has been nearly an order of magnitude reduction in the cost of wireless monitoring over the last 10 to 20 years.

4 PRACTICAL CONSIDERATIONS

4.1 Accuracy and robustness of sensors

While it is easy now days to find a variety of low cost, consumer grade, Internet-of-Things (IoT) devices for house-hold monitoring offered by Telecom service providers, electronics suppliers, systems integrators etc, these may not be the most appropriate devices to use in geotechnical or structural monitoring applications and particularly so when the monitoring is being used to control construction risks.

The accuracy requirements for most engineering monitoring, for example structural tilt, are usually quite onerous and are not able to be met by general consumer grade wireless sensors. Even when

comparing results for similar high quality sensors manufactured by well-established Instrumentation companies (see Plate 9 and Figure 1) it is not unusual to get varying results with different responses to environmental effects such as temperature changes, vibration and even electrical noise.



Plate 9: Tilt sensor comparison test

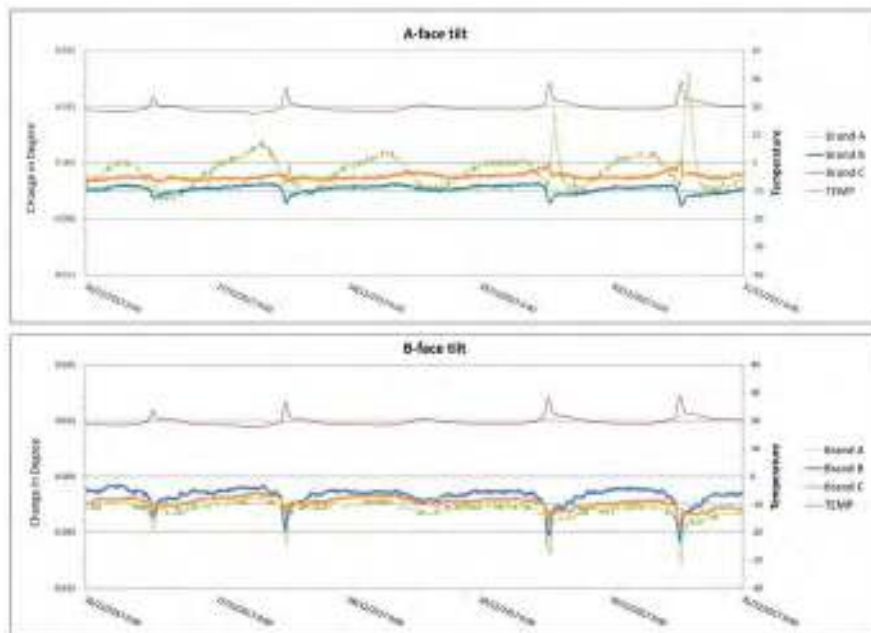


Figure 1: Tilt sensor comparison test results and temperature effects

4.2 Radio licensing requirements

Current wireless monitoring systems are usually designed to work over license-free radio frequency bands, but to be sure always check with the local regional regulations. In Hong Kong the Office of the Communication Authority publishes a table of frequency allocations which can be found at <https://www.ofca.gov.hk/filemanager/ofca/common/Industry/broadcasting>.

4.3 Installation locations

No single wireless monitoring system is likely to be suitable for use on every site and in all conditions.

LoRaWAN systems have very good data transmission distances but it is often a challenge to find suitable locations and stable power supplies for the gateways. This seems to be particularly true in the Hong Kong urban areas where tall buildings usually block radio transmissions and building owners are reluctant to allow installation of gateways for construction monitoring on their structures.

Mesh networks typically require more gateways than the long range systems but can often be battery powered, installed around the site and will route data between nodes so that although internode

distances are usually limited to 100m or less, data transmission distances of several 100m to gateways are still possible. Mesh networks are obviously not very suitable where there are few widely spaced monitoring points.

4.4 Robustness of data transmission and backup storage

Radio transmissions may often be blocked or fail so it's important that a wireless monitoring system has the ability to store and re-transmit data in order to minimize the loss of data. Frequent uploading of the gateway data to the Internet is also desirable but this may need to be balanced against power efficiency considerations.

4.5 Use of third party web services or software and vendor lock-in

Wireless monitoring systems may make use of third party Internet based services, software or messaging systems such as Google / Tableau / Telegram / WhatsApp to provide access to the monitoring data collected. In some cases these services may not be free for commercial use and in many cases they are not available in certain regions (for example anywhere in Mainland China).

Where it is not possible to directly access the wireless gateways to retrieve data or change configurations and it is necessary to go through a vendor provided service one needs to consider the risk of vendor lock in or what happens if the vendor closes down.

5 CONCLUSIONS

Wireless monitoring in Hong Kong already has a well-established track record which has been developed over the last two decades.

The cost of wireless monitoring systems has reduced by nearly an order of magnitude over the last 10 to 20 years to the extent where it is now more cost effective than manual monitoring whilst offering better quality, near real time readings on a 24 hour basis.

Sensor and the associated data acquisition technology has improved greatly, for example use of FFT based vibrating wire reading algorithms has virtually eliminated spike readings, so that consistent really good quality automated readings are possible.

The latest generation of wireless monitoring systems are robust and include built-in safeguards against data loss.

Automated wireless monitoring systems ensure comprehensive, reliable data are available allowing timely, informed Engineering decisions to be made improving risk management and even helping to optimize designs.

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Building and testing virtual rock

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ABSTRACT

The engineering behavior of rocks is often semi-unpredictable due to the varied mineralogical composition, textural properties and microstructures present. The strength and deformation parameters of rock are routinely obtained from the laboratory tests performed on intact rock specimens collected from the ground investigation works. Preparation of a large quantity of rock specimens and the repetitive laboratory experimental operations are very often quite time and labor demanding, which has motivated geotechnical practitioners to explore alternative practical approaches to physical tests. This research takes advantage of the state-of-the-art numerical techniques to develop a rock equivalent digitally, i.e. virtual rock, which makes it possible to test and observe the mechanical properties and behavior of the real rock at a low-cost and efficient way. Such a numerical simulation scheme is capable of capturing the microstructural characteristics of the real rock and reveal the macro-mechanical phenomena from a microscopic point of view, which is a promising supplement to routine laboratory tests.

1 INTRODUCTION

Rocks are found almost everywhere in nature. Most of the engineering activities, such as underground excavation, geothermal exploitation and nuclear waste disposal, require an in-depth understanding of the engineering behavior of rocks. However, reliable “prediction” of the mechanical properties and behavior of rocks is challenging, due to the varied mineralogical compositions, textures, microstructures and decomposition (weathering) state. In typical analysis and design in rock engineering, routine laboratory tests are performed to obtain the strength and deformation behavior (and design parameters) of rock. Some high-end technologies (e.g., μm CT scanning, serial thin section, digital image processing, etc.) have been increasingly applied to assist the observation and characterization of rock properties. These experimental studies increase our knowledge of the mechanisms of rock mechanical behavior in response to external loading. Performing physical tests, however, involves tedious procedures, including collection of rock cores from the ground investigation works, preparation of a large quantity of identical rock specimens and the repetitive laboratory experimental operations which are often time and labor demanding. In addition, investigating the influence of microscopic factors such as grain size distribution, mineral content, etc., as well as macroscopic factors such as planes of weakness on the deformation and fracturing behavior of rock is highly restricted in the laboratory environment.

Thanks to the advance in computer technology, numerical modeling of rock to predict and/or assess its field applicability under various geological settings have drawn considerable research effort. In the digital world, rocks are reproduced and tested in a low-cost and efficient way. The same rock can be tested repeatedly without being affected by laboratory operations or randomness of rock intrinsic properties, which is not feasible in the real word. Virtual rocks also offer significant convenience to study the influences of the testing condition that is hardly replicated or requires highly sophisticated instrument in laboratory, such as applying high confining pressures. Moreover, observation of microfracturing and failure process of rock is much favored in rock equivalent digitally, thereby revealing the micromechanism

accounting for the macro-mechanical behavior. Given the merits of virtual rocks, numerical techniques have been extensively developed to complement laboratory tests.

Realistically simulating brittle fracture has been one of the most critical issues in the geomechanical field (Mayer and Stead 2017). The success of using digitally virtual rocks to study the failure behavior of brittle rocks relies on a sufficient representation of the geometrical and topological properties at the grain scale (Potyondy 2015). Unfortunately, the grain structure cannot be exactly reproduced in any numerical model (Lan et al. 2010). Based on this consideration, this paper presents a robust crystalline rock model which is able to capture the grain-scale characteristics and simulate the macroscopic behavior from its microscopic mechanics. It also generates valuable insights into the transformation of the conventional geomechanics towards the digitization and the use of smart and innovative technology.

2 NUMERICAL TECHNIQUES FOR SIMULATION OF CRYSTALLINE ROCKS

2.1 Characteristics of crystalline rocks

It is difficult to characterize the crystalline rock (in fact any other rocks) because (1) the rocks may significantly vary in mineralogy, component content and structure, and thus in phenomenological behaviors; (2) the gross response to external loads is greatly influenced by many other external factors, such as temperature, confinement, and strain rate, etc (Horii and Nemat-Nasser, 1986). To evaluate the virtual rocks in a general treatment, the most recognized characteristics of crystalline rocks observed in laboratory should be noted.

(1) Highly interlocked structure associated with low porosity (Noferesti and Rao, 2010).

(2) Inherent microdefects such as cavities and cracks which serve as stress concentrators that promote micro-cracking (Horii and Nemat-Nasser, 1986).

(3) Non-linear stress-strain relationship characterizing microcracking process, as illustrated in Fig. 1. Generally, four stages of cracking evolution can be identified: (I) closure of initial microdefects, (II) initiation, (III) stable and (IV) unstable growth of new cracks (Diederichs, 2003). Two important parameters that characterize crack initiation (σ_{ci}) and damage (σ_{cd}) can be measured from the stress-strain curve, approximately 40% and 80% of the peak strength, respectively.

(4) Brittle failure mechanism is mostly controlled by tensile cracking, and shear cracking along grain boundaries only dominates near the peak strength (Nicksiar and Martin, 2014).

(5) Relatively low tensile to compressive strength ratio (around 0.03-0.04) (Hoek and Brown, 1998).

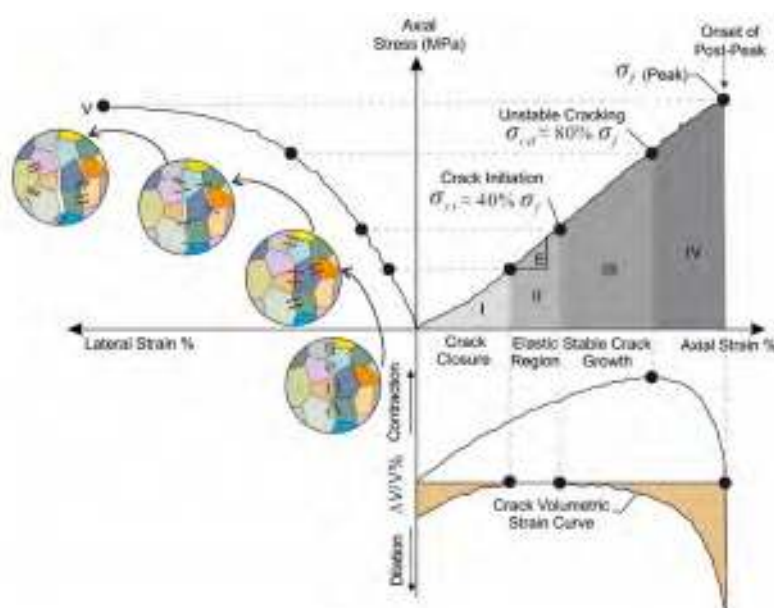


Figure 1: Illustration of brittle failure of crystalline rocks (Lac du Bonnet granite) by stress-strain relationship under uniaxial compression (after Martin and Chandler, 1994 and Cho et al., 2007). Micro-cracking behavior from closure of initial microdefects to initiation, stable and unstable growth of new cracks is well characterized by the non-linear stress strain curve.

2.2 Commonly used rock models

In the last decades, numerous numerical techniques have been developed aiming at solving rock mechanics problems at different scales. Generally, three categories of numerical methods are favorably used for simulating rock deformation and failure behavior: indirect methods based on continuum damage mechanics, e.g., the Finite Element Method (FEM) and Finite Difference Method (FDM), direct methods based on fracture mechanics principles known as the Discontinuum/Distinct Element Methods (DEM), and the hybrid Finite-Discrete Element Methods (FDEM).

Indirect approaches idealize rock as an equivalent continuous medium and damage is defined by constitutive relations (Matsui and San, 1992). At the large engineering scale, rocks may contain weak structural planes such as faults, persistent joints and bedding planes, in which case the strength and deformation of rocks are predominantly structurally controlled. While simulation of engineering problems is less demanding of the small-scale accuracy, indirect methods are widely used for achieving higher computing efficiency. However, this approach fails to provide an intuitive understanding of the local failure because material properties are mostly averagely measured. Therefore, most conventional continuum-based rock models are limited to account for the micro-scale and meso-scale mechanical problems such as progressive fracturing and fragmentation of rock.

On the contrary, direct methods make it feasible to model the progressive failure by treating the material as a collection of structural units (springs, beams, etc.) or separate bodies (particles, blocks, etc.) bonded at their contacts. The breakage of a bond creates a rupture surface, namely, a crack that can be easily related to the simulation of cracking process and acoustic emission (Khazaei et al. 2015). Such cracking occurrence is not governed by any predefined constitutive law, but “spontaneous” under the motion and interaction of bodies. In other words, emergent behavior of the numerical rock originates from and is directly related to microscopic interactions of discrete elements.

The hybrid FDEM is such a numerical method that the solid parts of a rock are represented as three-noded elements (i.e., triangles), and the discontinuities such as cracks are embedded between each pair of triangular elements as four-noded elements. The deformability of the solid parts is described using continuum mechanics principles such as linear elastic fracture mechanics (LEFM), while the behavior of discontinuities is explicitly defined by discrete element algorithms (Mahabadi et al. 2012). Mechanical heterogeneity of the rock can be modeled by assigning different mechanical properties (e.g., Young’s modulus and permeability) to the triangular elements conforming some statistical distributions. Within a triangulated grain, the nodes are not shared by adjacent triangular elements, thereby allowing the splitting of two neighbors. As such, an intact body can break via intra-fracturing or inter-fracturing, which is associated with the formation of sub-elements and crack elements between them. This method is effective for evaluating the overall behavior of rocks, but difficult to characterize the behavior of individual cracks (Jiang et al. 2009).

3 BUILDING VIRTUAL ROCK USING GRAIN-BASED MODEL

3.1 Contact mechanisms

Recognizing that the macro-mechanical behavior of rocks is governed by their grain-scale properties, the discrete element methods are more superior in capturing these salient fine features. The commercial software Particle Flow Code in two dimensions (PFC2D), which is developed based on the concept of DEM, is powerful and flexible in modelling the macroscopic rock behavior from its microscopic mechanics. A PFC2D material is represented as an assemblage of circular particles (i.e., disks) interacted at their contacts, termed the bonded-particle model (BPM) (Potyondy and Cundall 2004). The particles themselves are rigid and unbreakable, and only the contacts are allowed to break. When a contact breaks in the form of either tension or shear, a rupture surface forms as a tensile/shear crack. A parallel-bonded BPM is a packed granular assembly with all contacts using the linear parallel bond model, which mimics the mechanical behavior of a cement-like piece deposited between the two contacting particles.

Although the BPM approach offers unique advantages for explicitly simulating the entire cracking process, it suffers limitations in terms of capturing the effect of grain interlocking (Zhang and Wong 2018). To overcome the deficiencies of BPM, the grain-based model (GBM) was developed to incorporate

the geometrical complexity of mineral grains. The GBM in PFC2D (termed as GBM-PFC2D) refers to such a model that grain and grain boundary properties are mimicked by different contact models, namely, parallel bond and smooth-joint contact models, respectively (Potyondy 2010). Both parallel bonds and smooth joint contacts can break, corresponding to the intragranular microcracks and intergranular microcracks respectively. Fig. 2 illustrates the mechanical formulation of GBM-PFC2D.

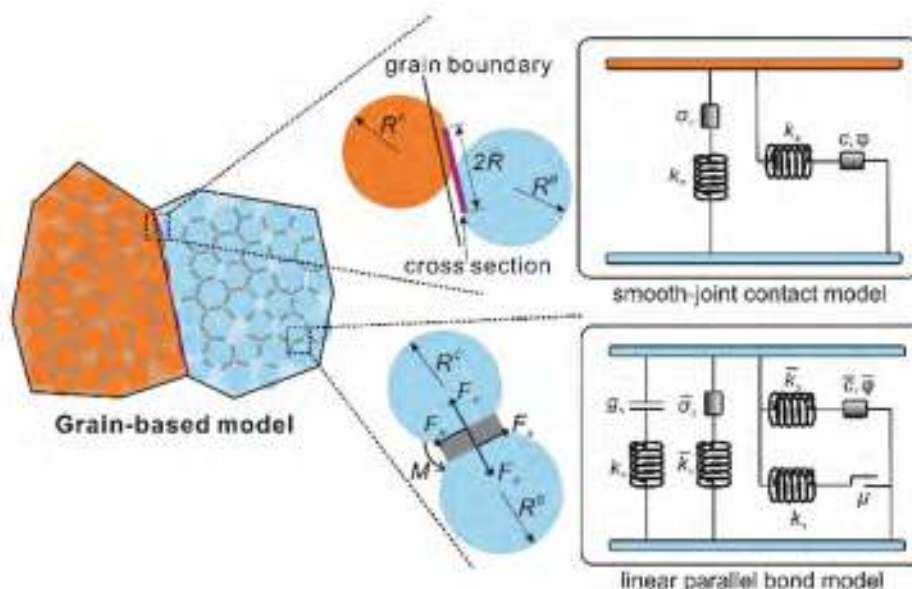


Figure 2: Mechanical formulation in GBM. Refer to Table 1 for the meaning of symbols.

3.2 Construction of rock model and grain structure

Following the framework of GBM-PFC2D (Potyondy 2010), the rock model generation consists of three steps: Step 1. Build a parallel-bonded BPM as the base model. Creation of a base material includes compacting initial assembly of randomly distributed particles, installation of specified isotropic stress, elimination of floating particles, etc. Detailed description of the material-genesis procedure is documented in the reference (Potyondy and Cundall 2004). Step 2. Overlay a grain structure on the BPM. Step 3. Substitute the parallel bonds along grain interfaces by smooth-joint contacts.

In step 2, a rock grain structure can be either statistically or realistically generated. In terms of the complex microstructural geometry of crystalline rocks, the Voronoi tessellation logic is robust in its potential to take the polycrystalline morphology into a computational form. The grain structure in GBM-PFC2D generally employs the Voronoi tessellated fabric (Potyondy 2010; Peng et al 2017), which considers the first order properties of the real morphologies: grain size distribution and the number of first neighbors. As an alternative to the statistical Voronoi algorithm, the realistic grain structure can be obtained by examining rock thin sections (Wong and Zhang 2018) or using the digital image processing technique (Zhang et al 2018). In order to investigate the influence of microstructures on the mechanical behavior of rock, three different 2D rectangular numerical rocks of the same dimensions (1.5 mm in width, 3 mm in height) with different fabrics are generated, namely, a homogeneous honeycomb structure (GBM-1), a heterogeneous Voronoi structure (GBM-2), and a realistic structure (GBM-3). The grain structure of GBM-3 is reconstructed from Carrara marble thin sections which comprise 421 complete grains in total. The three GBMs are shown in Fig. 3.

4 TESTING VIRTUAL ROCK

4.1 Microparameters of numerical rock models

In this study, the monomineralic crystalline rock Carrara marble is modeled. The microparameters for the three GBMs have been well calibrated by matching the laboratory response of the Carrara marble specimen with regard to the uniaxial compressive strength (UCS), Young's modulus (E) and

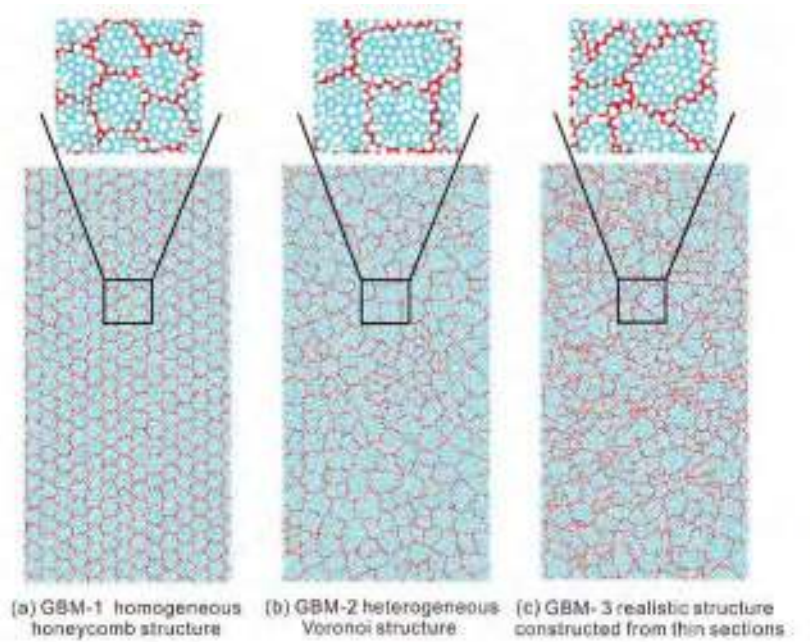


Figure 3: Microstructure configurations of the three GBMs. Green and red lines represent the parallel bonds (intra-grain contacts) and smooth joint contacts (inter-grain contacts), respectively.

the microcracking behavior. The values of UCS and E measured from uniaxial compression tests on cylindrical Carrara marble specimens are 88.3 MPa and 49.0 GPa, respectively (Alber and Hauptfleisch 1999). In particular, matching the microcracking behavior regarding the distribution, nature (shear or tensile) and type (intragranular or intergranular) of the microcracks takes into account the failure mechanism and the uniqueness of each rock specimen. For example, the development of tensile cracks is much more favored than shear cracks under no or low confining pressure. The calibration principles, assumptions and procedure are described in detail by Wong and Zhang (2018), and therefore not reiterated here. The calibrated microparameters are summarized in Table 1.

Table 1: Micro-mechanical parameters assigned to GBMs

Microparameters	GBM-1	GBM-2	GBM-3
Grain properties			
Minimum particle radius, $R_{\min}$ (mm)	0.15	0.15	0.15
Particle-size ratio, $R_{\max}/R_{\min}$	1.66	1.66	1.66
Particle density, ρ (kg/m ³)	2600	2600	2600
Contact normal to shear stiffness ratio, k_n/k_s	1.2	1.2	1.2
Particle-particle contact modulus, E_c (GPa)	95	78	
Particle friction coefficient, μ	1.0	1.0	1.0
Parallel bond radius multiplier, λ	1.0	1.0	1.0
Parallel bond normal to shear stiffness ratio, $\bar{k}_s/\bar{k}_n$	1.2	1.2	1.2
Parallel bond modulus, $\bar{E}_c$ (GPa)	95	78	
Parallel bond tensile strength, $\bar{\sigma}_c$ (MPa)	55	60	
Parallel bond cohesion, $\bar{C}$ (MPa)	55	60	
Parallel bond friction angle, ϕ (deg)	30	30	

Grain boundary properties (Smooth-joint contacts)			
Smooth-joint contact normal stiffness, k_n (N/M)	2.1×10^{13}	1.8×10^{13}	2.1×10^{13}
Smooth-joint contact shear stiffness, k_s (N/M)	2.0×10^{13}	1.7×10^{13}	2.0×10^{13}
Smooth-joint bond tensile strength, σ_b (MPa)	8.2	8	8.2
Smooth-joint bond cohesion, C_b (MPa)	120	100	108
Smooth-joint bond friction angle, ϕ_b (deg)	30	30	32
Smooth-joint bond friction coefficient, μ_b	1.2	1.2	1.2

4.2 Simulation of uniaxial compression tests

Uniaxial compression tests are simulated on the three virtual rocks to demonstrate the capability of GBM-PFC2D in capturing the mechanical behavior of crystalline rocks. By recording the stresses, the spatial and temporal evolution of microcracks, the micromechanisms of rock failure can be well interpreted. Fig. 4 shows the stress-strain curve as well as the temporal evolution of different types of microcracks in each rock model. The spatial distributions of microcracks regarding the type, location, and orientation are displayed Fig. 5.

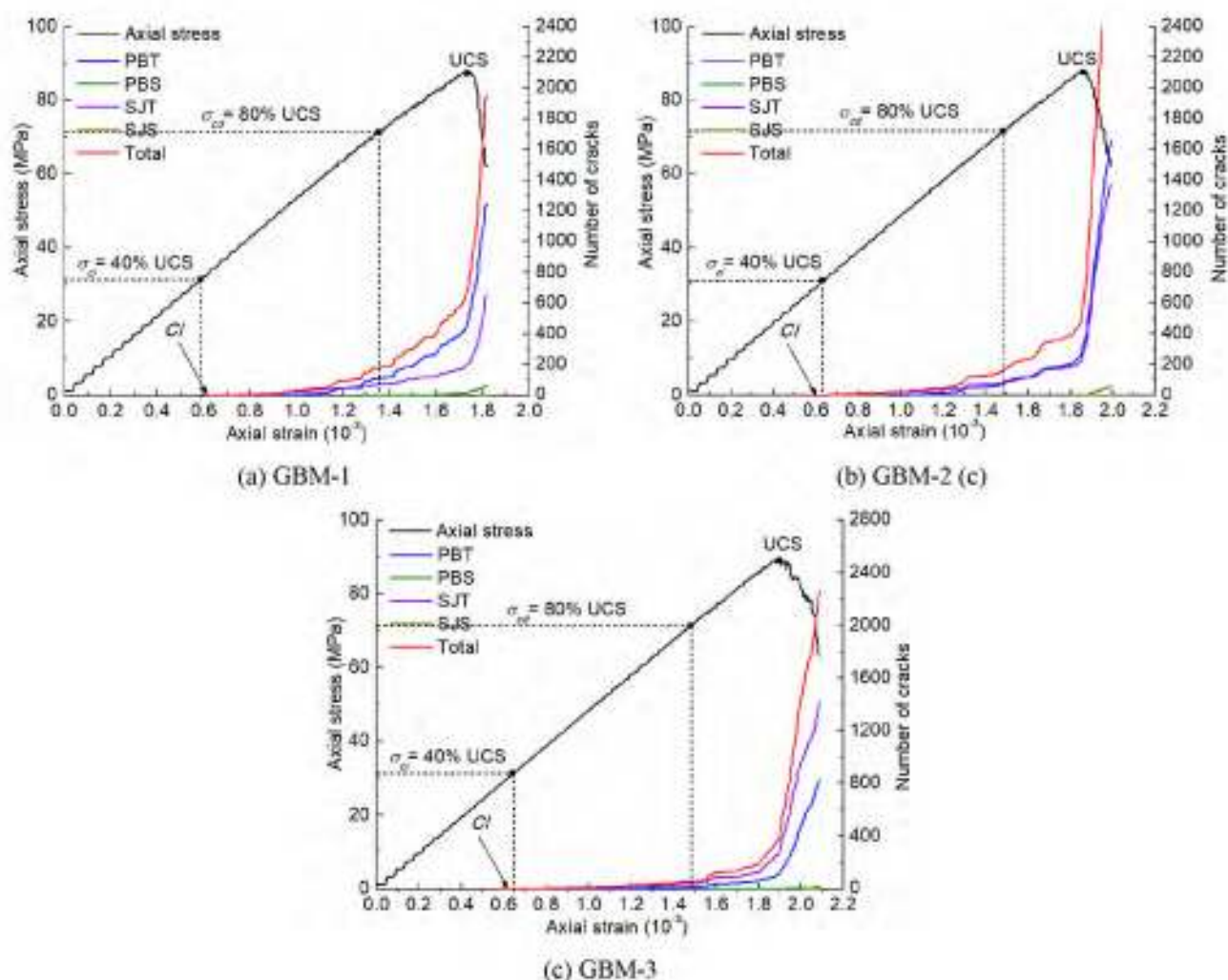


Figure 4: Evolution of micro-cracks of different numerical models under uniaxial compression. PBT, PBS, SJT and SJS refer to parallel bond broken in tension (intragranular tensile crack), parallel bond broken in shear (intragranular shear crack), smooth joint broken in tension (intergranular tensile crack) and smooth joint broken in shear (intergranular shear crack), respectively.

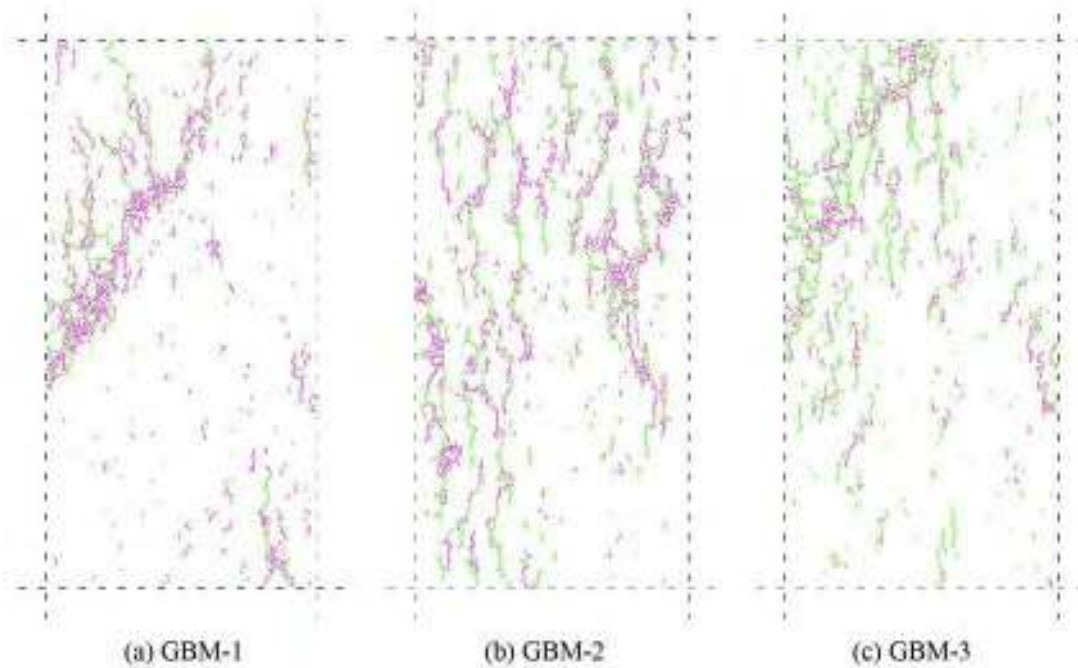


Figure 5. Distributions of microcracks in numerical rock models at 30% stress drop after the peak strength. Green and red refer to intergranular tensile and shear cracks, respectively; magenta and blue refer to intragranular tensile and shear cracks. The dashed lines denote the sample boundaries.

All the rock models undergo typical brittle failure, which is characterized by a linear stress-strain relationship in the pre-peak region followed by a post-peak strength drop. However, the nonlinearity of the stress-strain curve at the initial stage of loading due to the closure of initial microdefects is not well captured. It can be observed from Fig. 4 that the crack initiation points, σ_{ci} , of the three GBMs are very close to the suggested value of 40% of the peak strength (UCS), i.e., 41.57%, 37.5% and 35.6%, respectively. Microcracks grow slowly in the pre-peak stage but multiply upon approaching the peak strength. In fact, the onset of shear cracks is found to be in the vicinity of the peak point. For all the examined models, either inter or intragranular tensile cracks dominate among the generated microcrack types, and initiate before shear cracks. The observations above suggest that rock failure under no confining pressure is mainly induced by tensile cracking.

Fig. 5 demonstrates that the macroscopic rupture paths formed in the three models are different from each other attributed to different microstructures. When the microcrack density is sufficiently high, the interaction between cracks results in a macroscopic shear fracture in GBM-1. In the case of GBM-2, microcracks are predominantly oriented parallel to the axial load direction, indicating the axial splitting failure mode. GBM-3, however, presents both shear band and axial fractures, suggesting a mixed failure mode. As can be assessed by intuition, GBM-3 is the most geometrically heterogeneous. From a microscopic view, the ratio of inter to intragranular tensile cracks generally increases with the increase of microstructural heterogeneity from GBM-1 to GBM-3. In GBM-1 and GBM-2, intragranular tensile cracks are more preferred, while intergranular tensile cracks prevail over intragranular tensile cracks in GBM-3. The evidence suggests that increasing geometric heterogeneity intensifies stress concentration along their boundaries. As a consequence, the opening and sliding of grains is greatly promoted, which involves a more complicated microcracking process.

5 CAPABILITIES, LIMITATIONS AND PROSPECTS OF NUMERICAL ROCKS

The GBM-PFC2D is a promising numerical technique for its unique ability to simulate the macro-fracturing behaviour and macro-mechanical properties of highly interlocked rocks. The rock media is modeled as an assembly of particles interacting at their contacts in the virtual digital world. Each polygonal mineral grain in the rock model includes several bonded particles, while the particles adjacent to the grain interfaces can slide along grain boundaries. During the numerical loading test of the virtual rock,

both intra and intergranular contacts are allowed to break, whereby mimicking real grain crushing and grain boundary weakening accompanying the process of rock fracturing. By simulating uniaxial compression tests on three virtual rocks with different microstructures, the capability of GBM-PFC2D in mimicking rock brittle failure is demonstrated. Crack initiation, propagation and coalescence in the virtual rock are well visualized by recording the stresses, deformation as well as the temporal and spatial evolution of microcracks during rock testing, thereby the micromechanisms of the macroscopic fracturing phenomenon of the real rock counterpart can be revealed. As compared with other numerical rock models (e.g., the conventional bonded-particle model), GBM-PFC2D excels in the following four aspects regarding the characteristics of crystalline rocks as generalized in section 2.1:

(1) *Reproducing the crystalline microstructure associated with a high degree of interlocking.* An ideal polygonal structure with similar topological and properties of the physical specimen can be characterized using the Voronoi tessellation logic. Alternatively, a realistic microstructure can be used to represent monomineralic rocks (e.g. marble) based on the detailed examination of thin sections (Wong and Zhang 2018) or represent polymineralic rocks (e.g. granite) using the digital image processing technique (Zhang et al 2018).

(2) *Allowing breakage of individual grains to release localized stress.*

(3) *Distinguishing intra and intergranular microcracks by using different mechanical models for the intra- and inter-grain contacts.* Meanwhile, tensile or shear properties of the microcracks are recognized.

(4) *Capturing a reasonable ratio of tensile to compressive strength, and the nonlinear strength envelope.* This is not shown in the present study, while some other studies performing tensile tests and confined compressive tests on GBM-PFC2D have suggested its capability of simulating such mechanical properties (Hofmann et al. 2015; Azevedo et al. 2015).

However, due to various constraints (assumptions) in the numerical simulation world, no model is perfect as it always has to be simplified to help understand the complex real world. The major challenges of GBM-PFC2D include the following aspects:

(1) It has limited ability to incorporate inherent microdefects, hence leading to the absence of non-linear phase in the initial stress-strain curve due to the closure of initial microdefects.

(2) A GBM-PFC2D involves a number of microparameters which are not directly measured from the physical tests. Currently, determination of a unique set of representative microparameters remains a challenging issue in PFC simulation.

(3) The applicability of the proposed virtual rock to the engineering field scale is restricted attributed to the large number of particles required that are highly computation demanding.

(4) The presented virtual rock specimens and the constituent mineral grains are 2D, while the real entities are 3D.

The above mentioned capabilities and limitations of GBM-PFC2D provide valuable insights into geomechanics modelling. Because grain-scale structure governs the micromechanical behavior and in turn the phenomenological response of rocks, a representative virtual rock highly depends on the fidelity of microstructural mapping on the grain scale. This may involve 1) high-end techniques such as electron microscope scanning, CT scanning to characterize rock microstructures and mineral grain assemblage in detail; 2) computer technologies such as digital image processing, computer random simulations to transform the microstructural information into digital form; and 3) development of 3D rock models to reflect the realistic microstructural characteristics. On the other hand, natural rocks in the field are subjected to the interplay of in-situ ground stresses, underground fluid and ambient temperature, etc. The multi-physical coupling simulations considering the relevant environmental effects will be an important future direction of virtual rock development.

ACKNOWLEDGEMENTS

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Coupled analysis on optimizing installation configuration of energy dissipaters on flexible barriers

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ABSTRACT

Energy dissipaters, or alternatively, brake elements are installed onto cables of a flexible barrier to enhance the energy adsorption capacity of a barrier system. Various installation configurations of energy dissipaters can be found in a variety of barrier systems available on market, either for rockfall protection or flow-like debris resisting. They have been standardized for each specific product based on either static analysis or full-scale testing. With the advance of numerical method in carrying out coupled analysis of debris and/or boulder interaction with flexible barriers, performance of those installation configurations can now be evaluated cost-efficiently. This study investigates barrier deformation, cable force and elongation of energy dissipaters under impact of boulder in light of this background. Three types of configuration have been selected for study: Series in Two sides (ST), Parallel in Two sides (PT) and Parallel in One side (PO). It has been found that PT seems to be the most efficient arrangement among the three options and in all circumstances, it should be avoided to have a force-bearing cable without any energy dissipater.

1 INTRODUCTION

A typical flexible barrier system often comprises horizontal cables, side cables, ring nets, shackles that connecting rings with cables, energy dissipaters and anchors that providing lateral support. Flexible barriers installed on an open hillside would involve one more component: supporting posts (Kwan et al. 2014). Various configurations of assembling these components, depending on designed impact scenarios and site conditions, are provided by barrier manufacturers. These assembled flexible barrier systems are often evaluated based on pseudo static analysis and/or validated against field tests (Ferrero et al. 2015; Kwan et al. 2018; Wendeler et al. 2008). Recent numerical advances in explicitly modelling coupled behavior of flexible barriers with debris and/or boulders (Koo et al. 2017; Yu et al. 2018) have facilitated the evaluation of performance of flexible barriers. These coupled analyses provide cost-efficient supplement to expensive and time-consuming full-scale tests. It has been demonstrated through physical modelling tests (Ng et al. 2016) or fielding monitoring (Wendeler et al. 2008) that the energy dissipaters play a critical role in attenuating the dynamic impact. The objective of this study is to investigate effect of installation configuration of energy dissipaters on flexible barriers on overall barrier performance under single boulder impact. Various configurations, with reference to products available in market, were modelled using LS-DYNA and results are discussed.

2 WORKING PRINCIPLE OF ENERGY DISSIPATERS

Flexible barriers rely on complex interactions among individual structural components to attenuate the dynamic impact. When debris and/or boulder hit the flexible barrier, load transferred to cables would finally reach energy dissipaters typically installed at cable ends. An energy dissipater would limit the amount of force transferred to anchor foundations. It achieves this by large plastic deformation. In this way, impact energy is dissipated instead of otherwise stored by elastic components of a barrier such as cables. Figure 2 is a typical load-displacement curve of an energy dissipater. When threshold brake force reaches, plastic deformation occurs and the force in neighboring cables and anchors would be limited from further developing until a maximum elongation is reached, after which the dissipater gains the stiffness of nearby cables. Different manufacturers would adopt different shapes of energy dissipaters; however, the working principle is the same. A detailed analytical approach in modelling energy dissipater can be found in Brighenti et al. (2013).

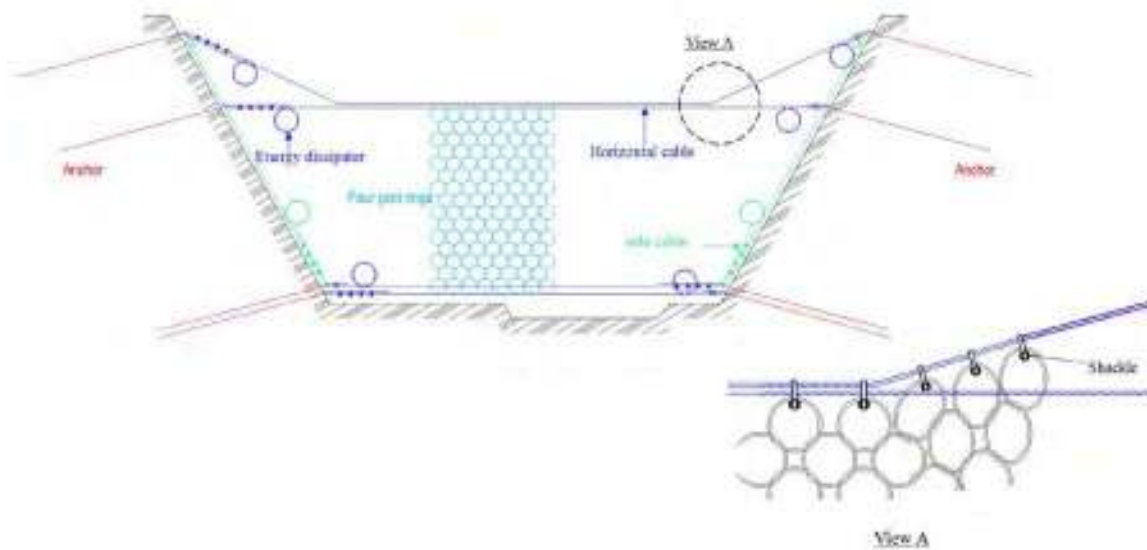


Figure 1: Typical configuration of a flexible barrier

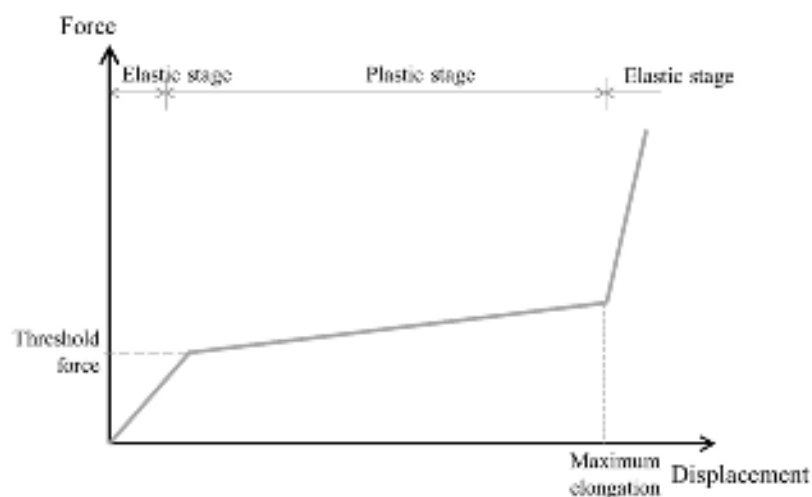


Figure 2: Load-deformation curve of an energy dissipater

3 COUPLED NUMERICAL MODELLING METHODOLOGY

3.1 Modelling of flexible barrier

Figure 3 shows the flexible barrier modelled. The rings, support ropes, and shackles were all modelled as elastic structures. Beam element was adopted to imitate these structures, except the pseudo 'membrane', which does not exist in a real flexible barrier. It was modelled using shell element. This pseudo membrane can be regarded as the fine mesh of flexible barriers installed on site. It helps to retain debris or boulders. The components of a flexible barrier modelled are summarized in Table 1.

Table 1: Material and element information for different parts of models

Part	Element type	Material model and parameters	Remark
Topography	Shell	Density = 2500kg/m ³ , E = 1×10 ⁷ Pa, Poisson's ratio = 0.3, Rigid (MAT020)	Thickness = 0.1 m
Boulder	Solid	Density = 2400kg/m ³ , E = 2×10 ⁷ Pa, Poisson's ratio = 0.2, Rigid (MAT020)	Diameter 1.5 m
<i>Following are modelling components of a flexible barrier</i>			
Cable	Beam	Diameter = 22mm, cross sectional area = 3.8×10 ⁻⁴ m ² , moment of inertia = 7×10 ⁻¹⁰ m ⁴ , E = 1×10 ¹¹ Pa, Poisson's ratio = 0.3, density = 7800kg/m ³ , Elastic (MAT001)	
Ring	Beam	Cross sectional area = 1.131×10 ⁻⁴ m ² , moment of inertia = 6.362×10 ⁻¹¹ m ⁴ , E = 2×10 ¹¹ Pa, Poisson's ratio = 0.3, density = 7800kg/m ³ , Elastic (MAT001)	
Shackle	Beam	Diameter = 19mm, E = 2×10 ¹¹ Pa, Poisson's ratio = 0.3, density = 7800kg/m ³ , Elastic (MAT001)	
Energy dissipater	Beam	Cable area = 3.8×10 ⁻⁴ m ² , volume = 1×10 ⁻⁴ m ³ , density = 2800kg/m ³ , stiffness coefficient = 5×10 ⁵ N/m, load-deformation curve, Inelastic spring discrete beam (MAT094)	See Figure 6 for Load-deformation curve
Pseudo membrane	Shell	Density = 2500kg/m ³ , E = 1×10 ⁷ Pa, Poisson's ratio = 0.3, Null (MAT009)	Thickness = 0.01 m

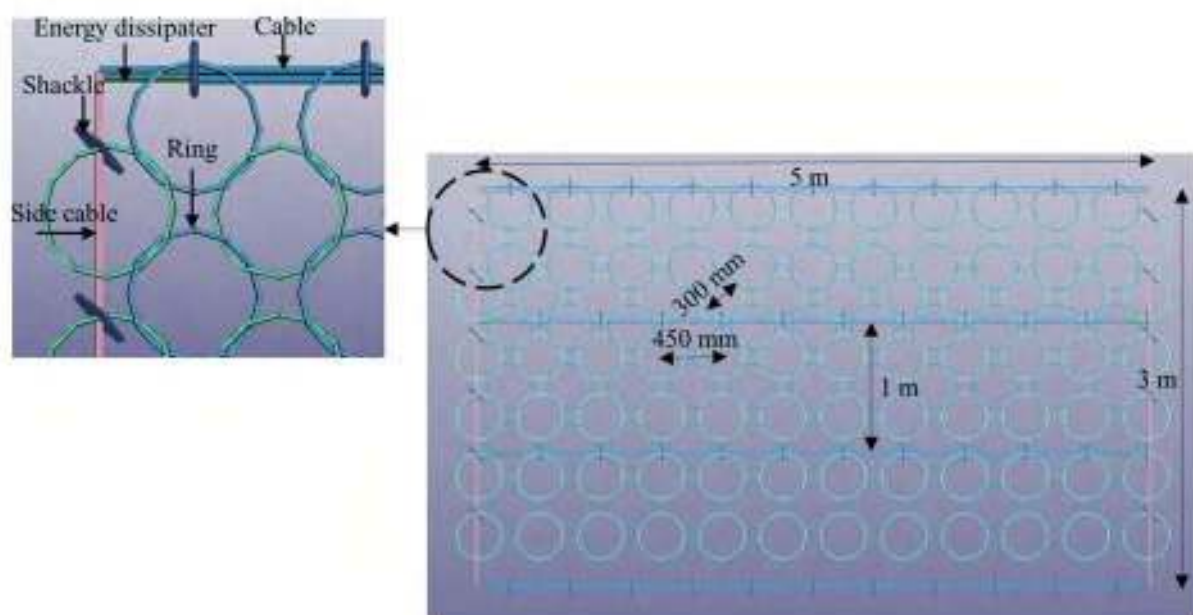


Figure 3: Model configuration of a flexible barrier

Connection between two individual rings is explicitly modelled. Each ring is connected to its adjacent ring by four joint points. Interaction would thus occur between rings. Shackles were used to connect the rings with adjacent cables. Thus, interaction would exist between them. It is also noted that during boulder impact, deformed adjacent ropes would interact with each other. To capture all these potential interactions, keyword “general contact” in LS-DYNA was adopted. Under this keyword, the master was defined as segment while the slave was defined as the components of flexible barriers (except pseudo membrane). A friction coefficient of 0.1 was assigned for all these interactions.

The end node of each level of cable or energy dissipater was fixed to simulate anchorage condition in field.

3.2 Modelling of boulder and its interaction with flexible barrier

Boulder impact on flexible barriers is the focus of current research. A rigid spherical boulder of 1.5 m diameter was modelled. It was located at 10m in front of the flexible barrier (Figure 4). Rigid material model was selected for the boulder. Element information regarding the boulder is summarized in Table 1.

To capture the interaction between the boulder and the topography, a friction angle of 12.5° was adopted. In addition, it is anticipated that interaction between boulder and barrier would occur. Contact between the boulder and the pseudo membrane need to be established. Keyword ‘contact surface to surface’ was used. A nominal friction coefficient 0.01 was adopted for this interaction so as to minimize the effect of pseudo membrane on load transfer.

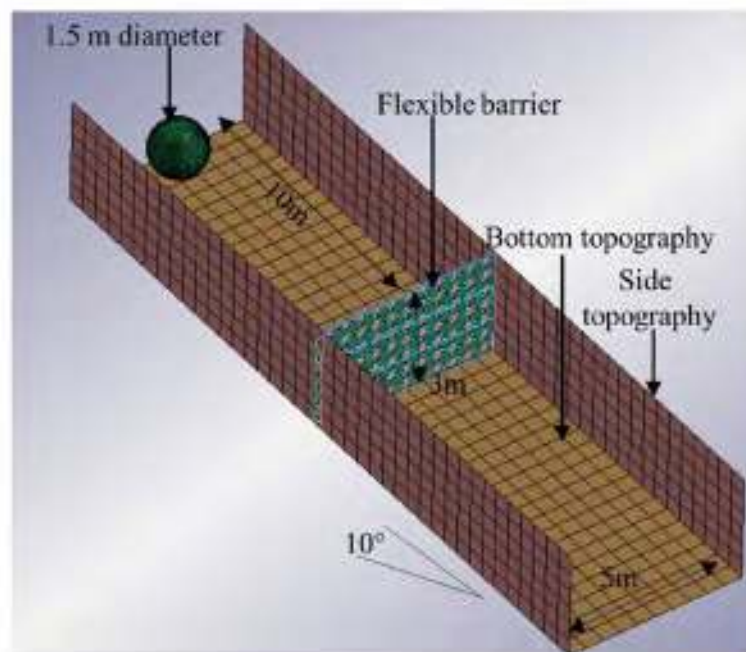


Figure 4: Model setup simulating a boulder impacting a flexible barrier

3.4 Numerical modelling programme

In total six cases were simulated. They were categorized according to installation configuration of energy dissipaters. For all cases, the total number of energy dissipaters in each level of horizontal cables is two and the number of horizontal cables in each level is two. As a result, three groups of configurations would exist. For Cases 1 and 2, the arrangement of energy dissipaters is termed as Series in Two sides (ST), as schematically shown in Figure 5(a). Parallel in Two sides (PT, Figure 5(b)) was adopted for Cases 3 and 4. In the remaining cases (Cases 5 and 6), Parallel in One side (PO, Figure 5(c)) applies. The load-displacement curve adopted is shown in Figure 6. The threshold force is 50 kN. For each installation arrangement, different impact velocities, i.e. $\sim 6\text{m/s}$ and $\sim 10\text{ m/s}$ were studied. Table 2 presents the numerical modelling programme.

3.5 Numerical modelling sequence

The model was subjected to gravity loading. Keyword “Load body Z” was adopted in LS-DYNA. The gravity was turned on at $t = 1 \times 10^{-4}$ s, almost immediately when boulder travelling started. After that, gravity was kept constant. Depending on each specific case, initial velocities of 5 m/s or 9 m/s were applied to the boulder.

Table 2: Analysis programme

Case	Energy dissipator configuration*	Impact velocity of boulder (m/s)
1_ST_6	Series in Two sides	6
2_ST_10		10
3_PT_6	Parallel in Two sides	6
4_PT_10		10
5_PO_6	Parallel in One side	6
6_PO_10		10

*Total number of energy dissipaters in each level of cables is kept to be 2. Each level has two horizontal cables.

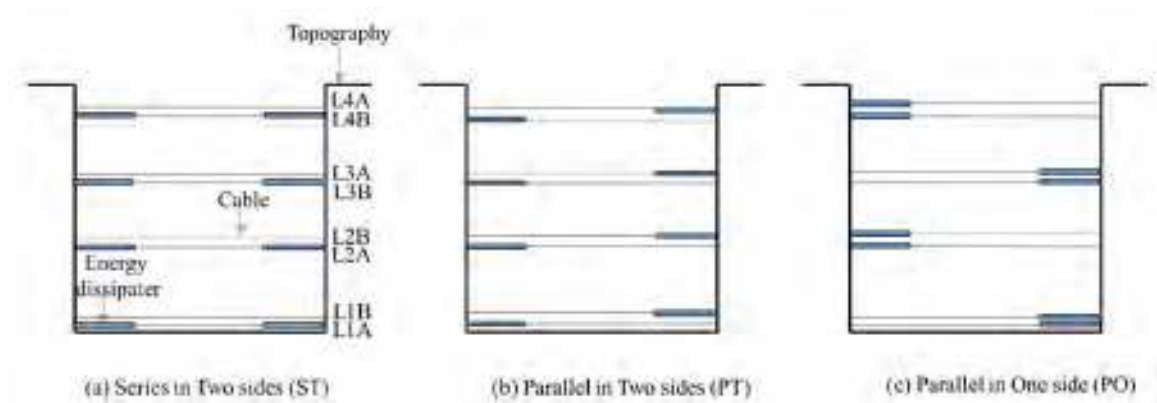


Figure 5: Schematic installation configuration of energy dissipaters on cables (other components are not shown for simplicity)

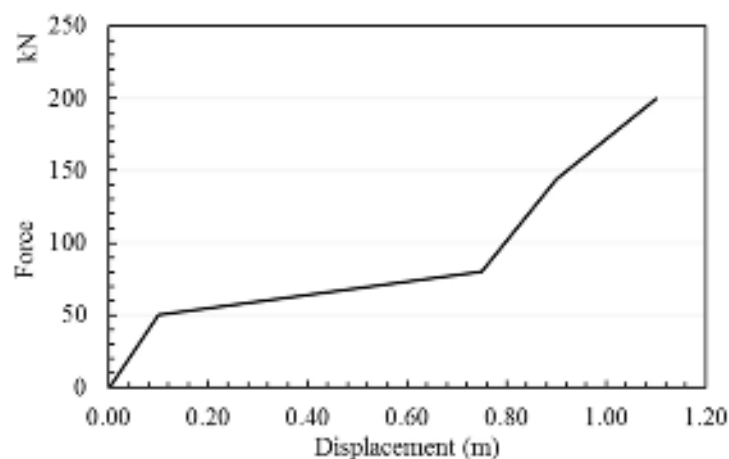


Figure 6: Load-displacement curve adopted in models

4 RESULTS AND DISCUSSION

Response of boulder before and after impacting the flexible barrier is shown in Figure 7. For cases with an initial boulder velocity of 5 m/s, the impact velocity is 6 m/s. Upon interacting with the flexible barrier, velocity reduced and gradually the boulder reached a full stop. However, in Case 1_ST_6, apparent boulder rebound was recorded compared with other two cases (Cases 3_PT_6 and 5_PO_6). This reflects when the installation configuration of energy dissipaters is series in two sides leaving one cable without any energy dissipaters, the barrier behaves stiffer. When the impact velocity increased to 10 m/s, extreme element distortion occurred for this type of arrangement due to load concentration and this impeded further computation. Essentially this implies the risk of localized damage of a flexible barrier under high impact energy scenarios. On the other hand, for all impacting velocities studied, boulder response is similar regardless energy dissipater is installed parallel in two sides or parallel in one side.

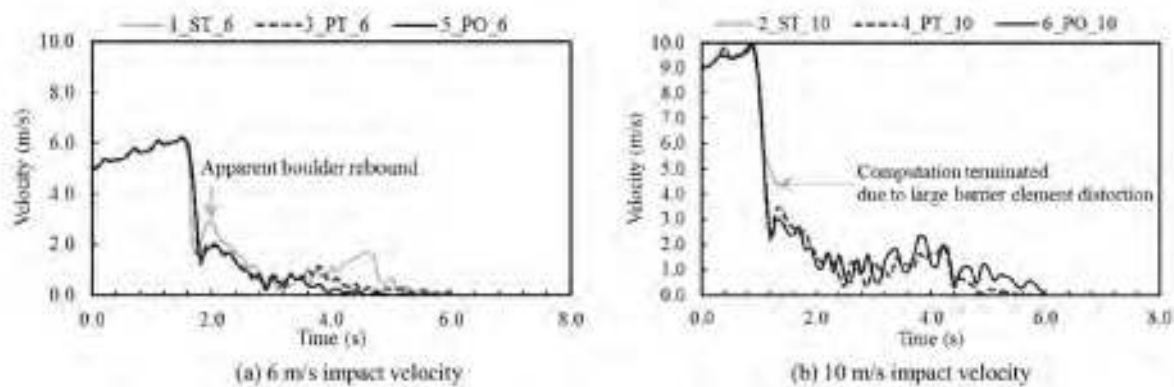


Figure 7: Boulder velocity before and after impacting the flexible barrier

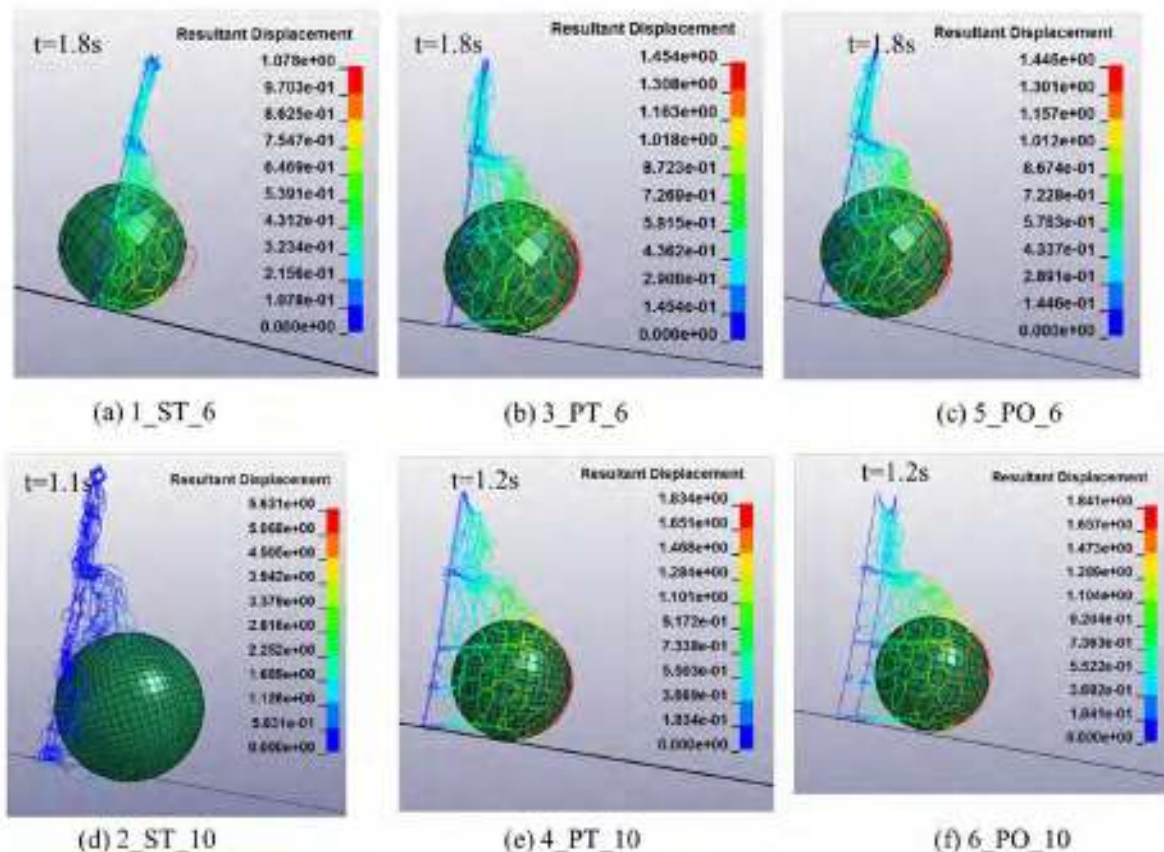


Figure 8: Maximum barrier deformation under boulder impact (unit: m)

Maximum barrier deformation is shown in Figure 8. For impact velocity of 6 m/s, maximum deformation occurs at $t = 1.8$ s for all cases. However, a barrier with energy dissipater installed series in two sides would give smallest deformation (i.e. most stiff). While for other two configurations, the maximum deformation is similar. For impact velocity of 10 m/s, the barrier with energy dissipater installed as series in two sides cannot effectively retain the boulder (extremely large localized barrier deformation around 6 m occurred), which means localized barrier failure would probably occur if this impact scenario happens in field. However, this was not the case for other two types of installation configuration wherein boulder was arrested. Again, configurations with parallel in two sides and parallel in one side do not give any difference in terms of barrier deformation, both resulting a maximum 1.8 m deformation at the time of 1.2 s.

Axial force developed in cables during impact is shown in Figure 9. For all cases, bottom two levels of cable were severely subjected to the impact force. Note that the distance between each level of cable is 1 m and the boulder has a diameter of 1.5 m. Thus, it is reasonable that the mobilized force concentrated on these two levels. However, it is noted that cables at L2 mobilized higher force than at L1. However, for the ST cases, load was concentrated on the cable without energy dissipater and as high as 400 kN was developed, regardless of the impacting velocity. This is almost the capacity of typical cables adopted in flexible barriers for site installation (Wendeler et al. 2008). With the introduction of energy dissipater in every individual cable (either installed parallel in one side or parallel in two sides), the maximum cable force reduced to 80 kN.

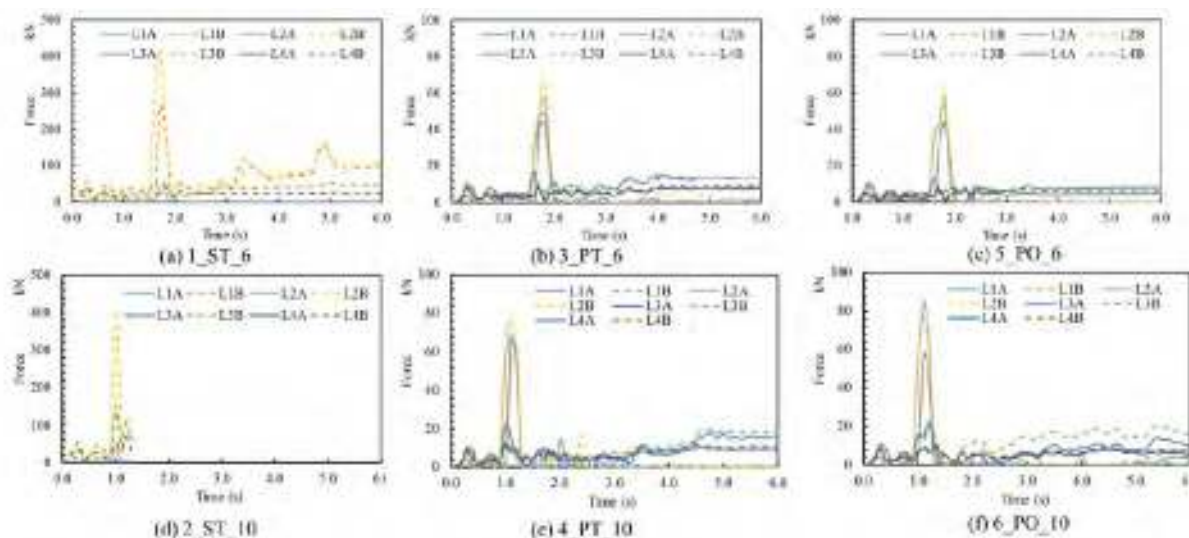


Figure 9: Cable force developed during boulder impact

Elongation of energy dissipaters of Case 2_ST_10, Case 4_PT_10 and Case 6_PO_10 is shown in Figure 10. In Case 2_ST_10, energy dissipaters did not have chances to mobilize during impact. As revealed by axial force described above, on each level of cables, load was concentrated on the other cable without energy dissipater. On the contrary, energy dissipaters were activated immediately when boulder impacted barrier in other two cases. In these two cases, energy dissipaters installed at L2 level elongated more than their bottom (i.e. L1) counterparts. Centrifuge modelling tests reported in Song et al. (2017) also recorded such pattern. It is also interesting to notice that when energy dissipaters were installed parallel in two sides (i.e. Case 4_PT_10), they were more uniformly utilized as revealed by the smaller elongation difference between energy dissipaters installed in cable level L1 and L2, compared with that in Case 6_PO_10 where energy dissipaters were installed parallel in one side. The implication of this to engineering practice is that it is better to average energy dissipaters in two sides. The added benefit is it would mitigate the risk they may be prevent from being activated in one go if they are concentrated in one location due to local incidents, such as boulder or debris directly hit them or bury them before they are to function (Kwan et al., 2014).

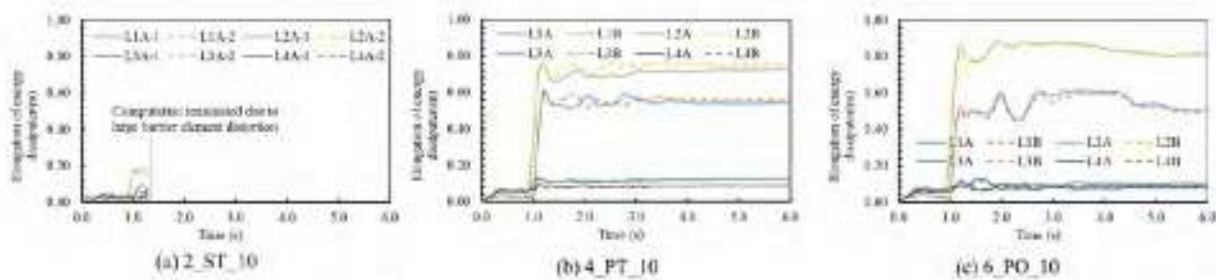


Figure 10: Elongation of energy dissipaters

5 CONCLUSIONS

Performance of a flexible barrier with various installation configurations of energy dissipaters under impact of boulders was studied. It is noted that for boulder impact, instead of the most bottom cables, cables at immediate higher level are prone to impact load. More importantly, to efficiently mobilize energy dissipaters, they should be evenly allocated in each force-bearing cable and in each side. Failing to comprise with this criterion would result in either load concentration or overuse of some individual energy dissipater. The current numerical analysis demonstrates the powerfulness of coupled analysis in dealing with flexible barriers, of which individual structural component can be explicitly and precisely studied. The observation made from this study may shed light on cost-effective implementation of natural terrain hazard mitigation works where flexible barriers are involved.

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Automatic Digital Monitor for Standard Penetration Test in Ground Investigation in Hong Kong

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ABSTRACT

This paper introduces the automatic SPT monitor, an in-situ digital device for automatic monitoring and recording of the standard penetration test (SPT) in ground investigation in Hong Kong. The SPT monitor has a SPT Tip-depth monitor and a SPT N-value monitor. The SPT tip-depth monitor checks and records the depth of the split spoon sampler during this inserting or retrieving from the borehole. The SPT N-value monitor checks and records both the shock axial force in drill rod and the shock axial penetration of the split spoon sampler in borehole induced by each of the hammer blowing in real time series. It can produce a test report for the N-value (N) and other factual data according to a given specification in real time and digital format. The device is portable and can be non-destructively mounted onto existing SPT apparatus. It can be used in normal ground conditions (i.e. $1 \leq N \leq 200$), extreme hard (i.e. $N > 200$) or extreme soft (i.e. $0 < N < 1$) ground conditions. The most weak and/or soft soil seam thinner than 75 mm to 300 mm can be detected from the monitored and recorded shock force and shock penetration per hammer blow.

1 INTRODUCTION

In 2003, Department of Civil Engineering of The University of Hong Kong (we) was awarded a financial support from the Housing Authority Research Fund (Agreement No. CB20030021) for the research project entitled "Automation of Measurement Monitoring / Recording for Standard Penetration Tests in Site Investigation". At that time, we were carrying out a series of research projects for landslide prevention including the automatic drilling process monitoring project (Yue et al. 2002; 2004). From June 2003 and May 2005, we carried out the research project for the development of a prototype equipment to automatically monitor SPTs and record the test results. We invented, designed, fabricated, verified and validated an integrated in-situ digital system (namely, automatic SPT monitor) for automatically recording and measuring the entire testing process of the Standard Penetration Test (SPT). During the two-year period, we received an in-kind support from the Housing Department of Hong Kong SAR Government. This automatic SPT monitor was received a USA invention patent in 2008 (Yue et al. 2008). The monitor is presented in the ensuing.

2 BACKGROUND AND MOTIVATION

2.1 Standard Penetration Test

As shown in Plate 1, the standard penetration test (SPT) is an in-situ test technique for soil resistance. The dynamical penetration of a split spoon sampler into the soil at the bottom of a drillhole is a failure process of the in-situ soil under loading, which is related to the soil strength properties. The sampler

penetration is forced by the impact blow of a steel hammer onto the drill rod connected to the sampler. The SPT is carried out by two operators in association with a drilling rig at field.

As shown in Plates 1 and 2, a SPT device has the following main components. The steel hammer has a mass of 63.5 kg and is freely falling down for a height 760 mm between the upper anvil and lower anvil along the steel rod. The upper and lower anvils confine the up and down movements of the hammer along the steel rod.

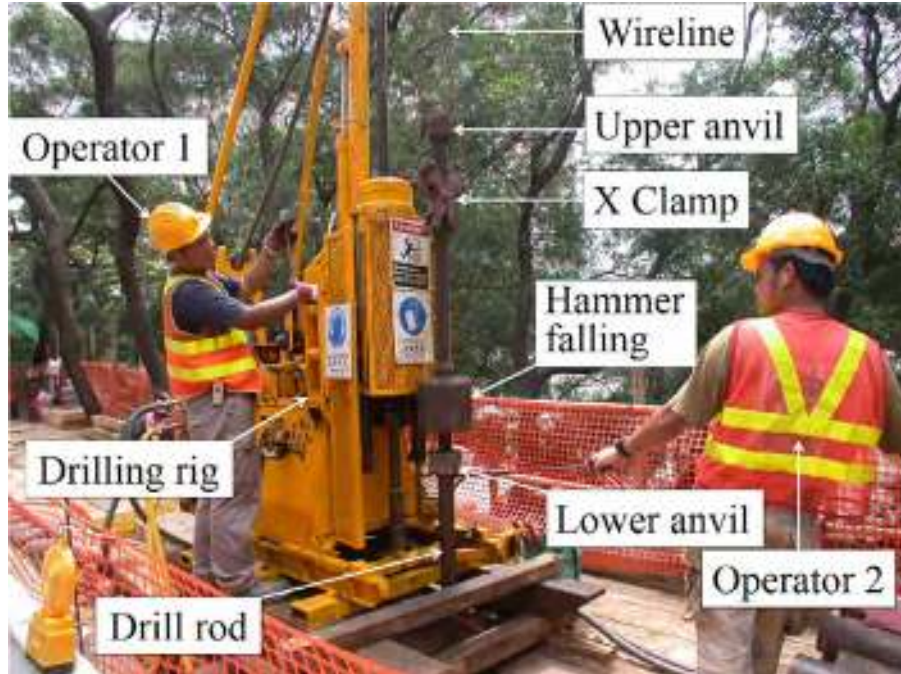


Plate 1: Operation of a SPT test in association with a drilling rig in Hong Kong

The split spoon sampler has a coupler at head, a split barrel in the middle and a driving shoe at the toe. It penetrates into the soil ground and inserts soil in its hollow tube. It connects to the lower drill rod, and the drill rods with male and female threads are connected and extended borehole. These components are attached to a drill rig for uplifting power and testing operation.

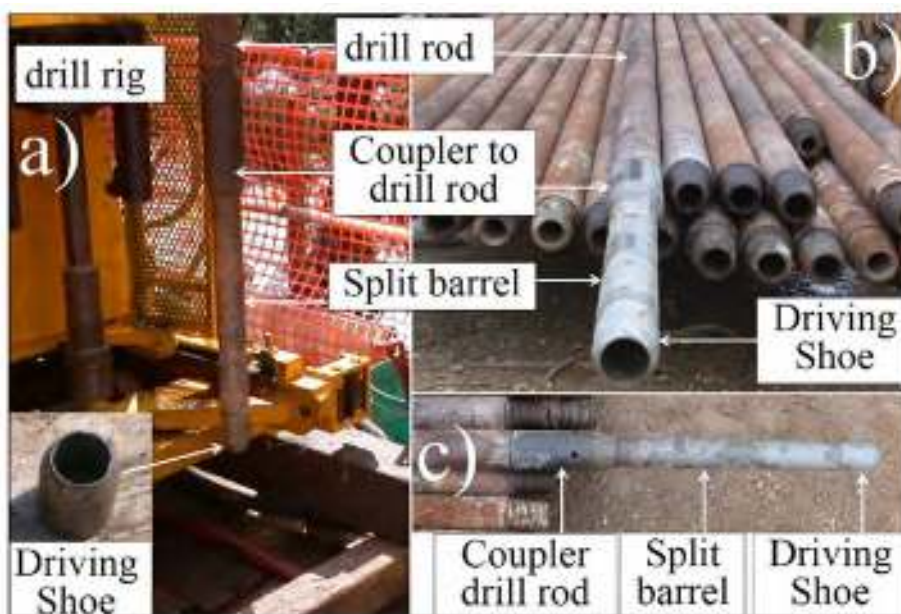


Plate 2: SPT split spoon sampler and drill rod

The operator 1 in Plate 1 uses the drilling rig to uplift and release the hammer via the wireline. The operator 2 mounts the SPT device onto the drilling rig and releases it after testing. The hammer is used to blow and force the sampler to penetrate into soil.

2.2 SPT Results

Plate 3 shows the measurement of SPT and test results. Plate 3a shows the manual measurement of the test depth in a borehole. Plate 3b shows the six divisions of the sampler penetration depth on a drill rod marked by the operator 2. Each zone is 75 mm in length. The total penetration is 450 mm (6×75 mm). Plate 3c shows the number of the hammer blows for reaching the 75 mm penetration depth of each of the six divisions.

According to HKHA (2003), the sum of the first and second blow numbers for the first and second 75 mm penetration divisions with a total penetration depth of 150 mm is the seating value in SPT, which is 12 (i.e. $2+10$) in Plate 3c. The sum of the third to the sixth blow numbers for the third to the sixth 75 mm penetration divisions with a total penetration depth of 300 mm is the SPT N-value, which is 27 (i.e. $10+5+4+8$) in Plate 3c.

Plate 3d shows the split barrel sampler with small amount of soil samples on the two ends. Plate 3e shows a white colored aluminum tube with sealed two ends containing a slightly disturbed soil sample. The tube is located within the split barrel in Plate 3d. Plate 3f shows the soil sample at the one end of the tube.



Plate 3: Manual measurements of SPT tests: depth in drillhole split spoon sampler and drill rod

2.3 Brief History, Standardization and Utilization

Earlier SPT development history can be found in Terzaghi and Peck (1948). Since then, SPT has been standardized in many countries and regions (ASTM 1986; 1999; BSI 1990; HKHA 2003; MOC 2001). It is the most routinely and frequently used in-situ probing test method for measuring soil properties and strength in ground investigation around the world including Hong Kong (GEO 1996; 2006).

The soil sample that is obtained as a part of the test (Plate 3d to 3f) can be used for water contents determination, soil type identification, and laboratory tests. The SPT blow-count, N , is a measure or index of the in-place firmness or density of the ground/foundation material. Many engineers have developed empirical correlation equations for using SPT N-value to determine the values of soil density, soil internal frictional angle, soil deformation modulus and soil bearing capacity in sands and general soil grounds (Yang and Yue, 2008).

2.4 The Manual Measurement Issue and Quality Need of SPT Works in Hong Kong

The SPT results, however, are obtained from the manual measurements of the two operators as shown in Plate 1. The manual measurements include the penetration depth for each blow, the number of blows for each of the six 75 mm penetration depth zone, the depth of the split-barrel sampler in borehole. At many occasions, there are no independent supervisions and inspections. Furthermore, the test is destructive, non-repetitive and time consuming. The test is often carried out in colluvium and weathered soils in Hong Kong. The possible random presence of gravels, cobbles and boulders in colluvium and corestones

in weathered soil makes it difficult to verify and non-accept the test results (i.e., the N-values) with or without significant and random variations in Hong Kong.

For example, a very wide range of the N-values between 8 and 100 was obtained at one of the sites in Hong Kong with weathered soil. Such the large variation, and therefore, the results could not be validated or verified. Hence, the accuracy and quality of the SPT results are always a big concern for many geotechnical engineers in Hong Kong. There is a need to have a tool to independently check and verify the accuracy and quality of the SPT test results, and this issue has to be resolved.

2.5 Objective of This Paper

To address this issue, this paper presents an in-situ digital device (i.e., Automatic SPT Monitor) for automatic monitoring and recording of the standard penetration test (SPT) in ground investigation in Hong Kong. The device monitors and records the shock axial force due to each blow of the SPT hammer onto the drill rod and the shock penetration of the SPT split spoon sampler induced by the hammer blowing in drillhole in real time series. It can produce a test report for the N-value and other factual data such as peak shock force and energy efficiency, according to a given specification in real time and digital format. The device is portable and can be non-destructively mounted onto an existing SPT apparatus.

3 THE AUTOMATIC SPT MONITOR

3.1 General

As shown in Figure 1, the SPT Monitor consists of a set of hardware and software. It automatically measures and analyzes SPT tip depth, SPT hammer blow count, SPT penetration depth, SPT axial force per hammer blow and SPT N value. The hardware comprises digital recording devices that can be nondestructively mounted on a SPT apparatus to measure various parameters in real time during testing. It consists of two sets of in-situ devices namely “SPT Tip-depth Monitor” and “SPT N-value Monitor”.

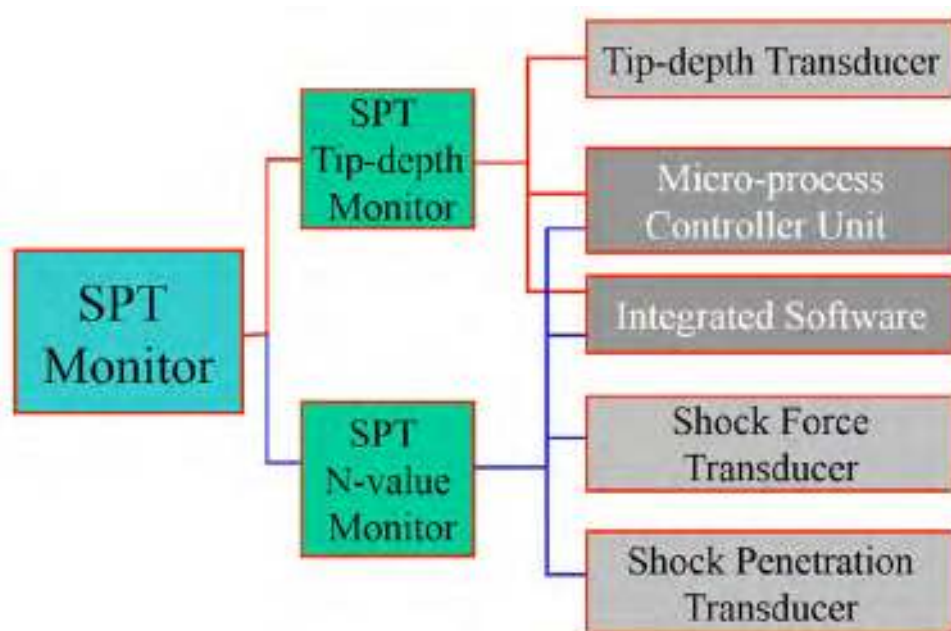


Figure 1: Components of automatic SPT Monitor

The software is able to control the SPT components to acquire and save data. It also has capability of accurately producing the SPT results from those acquired data and assessing the efficiency of transfer of the kinetic energy of the hammer blow into shock stress/force wave energy of the drill rods. The utilization of the Tip-depth monitor and the N-value monitor is given in Figure 2 below in association of the drilling work and SPT operation process in real time

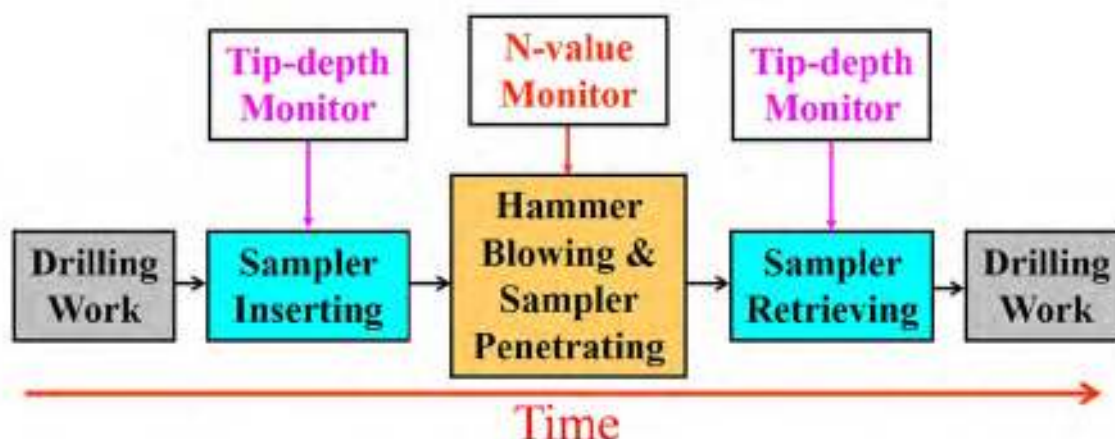


Figure 2: Utilization of the SPT Tip-depth Monitor and the SPT N-value Monitor During SPT Operation Process

3.2 The SPT Tip-depth Monitor for Processes of Sampler Inserting and Retrieving

The automatic Tip-depth monitor is shown in Plate 4, as it is mounted onto a SPT apparatus. It can automatically measure and record the inserting (or retrieving) process of the drill rod and split spoon sampler system into (or out of) a borehole immediately before (or after) an individual test in real time. During these two operational processes, it can accurately measure the depth of the sampler and the operation process. An example is shown in Figure 3. The details can be found in Yue et al. (2008) and Yang et al. (2012).

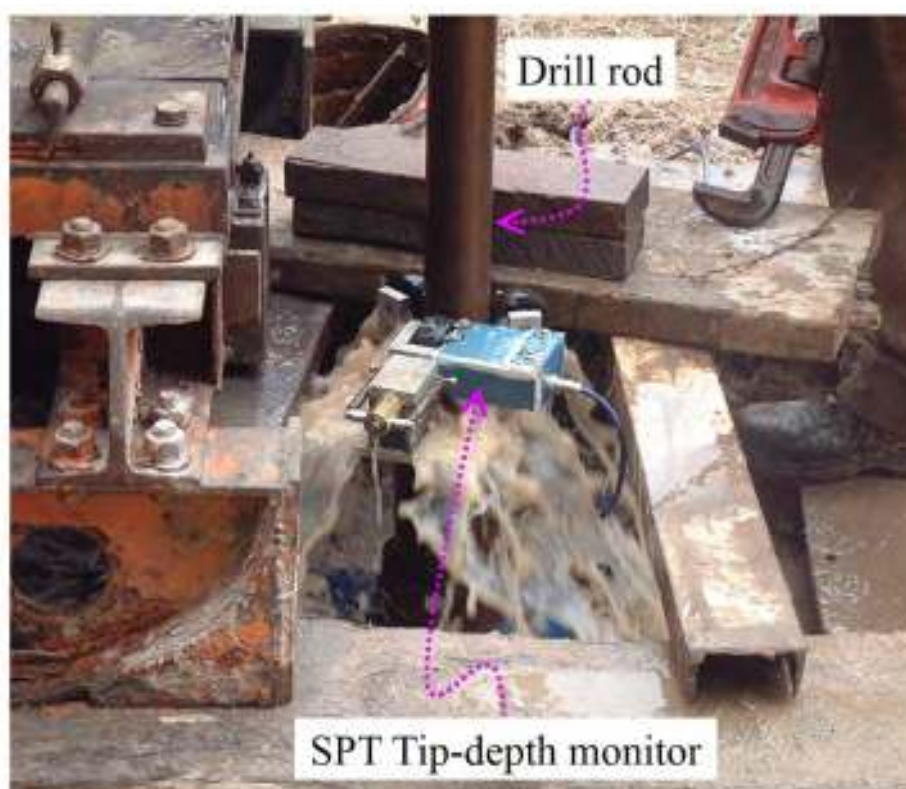


Plate 4: Operation of the automatic SPT Tip-depth Monitor in borehole at field

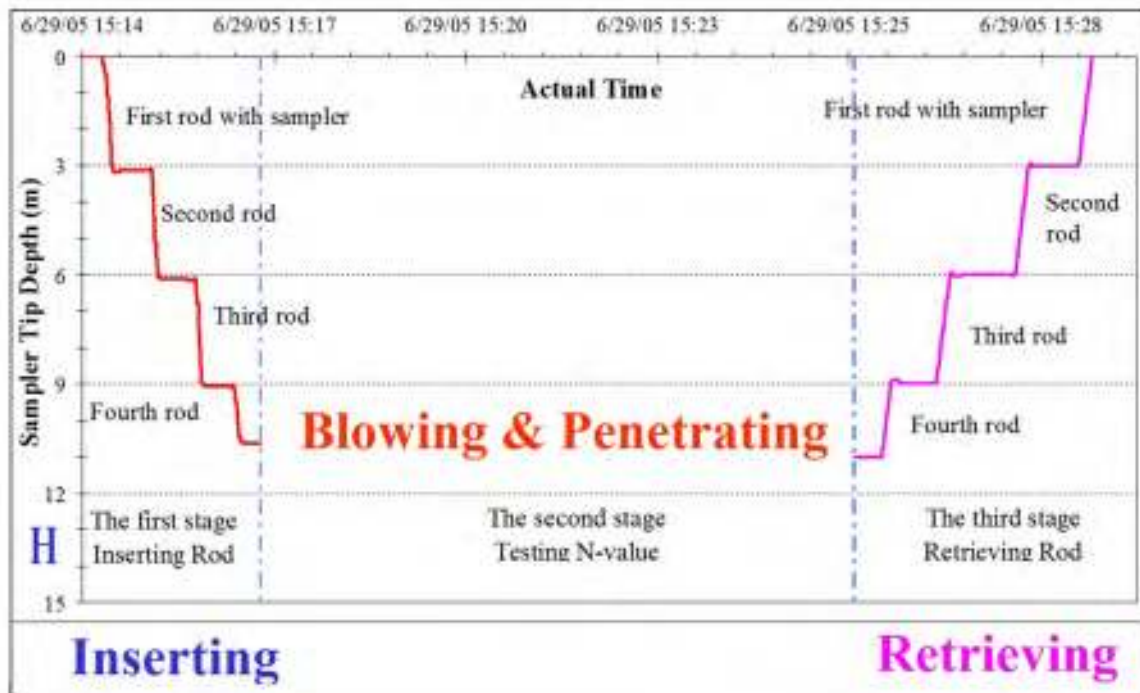


Figure 3: Result of the automatic SPT Tip-depth Monitor in borehole at field



Plate 5: The automatic SPT N-value Monitor in operation at field

3.3 The SPT N-value Monitor for Process of Hammer Blowing and Sampler Penetrating

The SPT N-value Monitor is shown in Plate 5 above, as it is mounted onto a SPT apparatus. It has a shock axial force transducer and a shock axial penetration transducer. The two transducers can respectively and automatically monitor and record the shock axial force in the drill rod and the shock axial deformation of the drill rod induced by each drop blowing of the steel hammer. The two results of the shock axial force and the shock axial deformation induced by one hammer blow are shown in Plates 6a and 6b, respectively. The shock axial deformation includes the instantly elastic recoverable axial shortening and elongation of the drill rod and the permanent penetration of the sampler into the ground below the borehole base. To avoid miscounting of the blow number, a criterion of minimum shock axial force of 50 kN is adopted. In other words, if the shock axial force is less than 50 kN, the SPT N-value Monitor would consider the hammer drop has a short vertical distance and do not account such hammer drop as an actual blow adding to the N-value.

The automation of the N-value measurement in real time is realized via the real time measurement of the shock axial force and the shock axial deformation by each individual hammer blow. The SPT sampler penetration at the j-th hammer blow is obtained by summation of the permanent penetrations induced by the first to the j-th hammer blow. An example is given in Plate 7 below. The measured axial force in the SPT rod can be further used to calculate the dynamic elastic deformation energy in the drill rod for the investigation of the SPT hammer efficiency. More details can be found in Yue et al. (2008).



Plate 6: The measured results of the automatic SPT N-value Monitor by one hammer blow at field

3.4 The Integrated Software

The integrated software does all the monitoring, recoding and calculation in real time during each hammer blowing. The Plates 6a, 6b, and 7 are mainly the screenshots from the SPT N-value Monitor. Once the total penetration depth reaches 450 mm or the total number of the hammer blows reaches 200, the Monitor would automatically give signals indicating the SPT test has reached the specification in Hong Kong (HKHA, 2003).

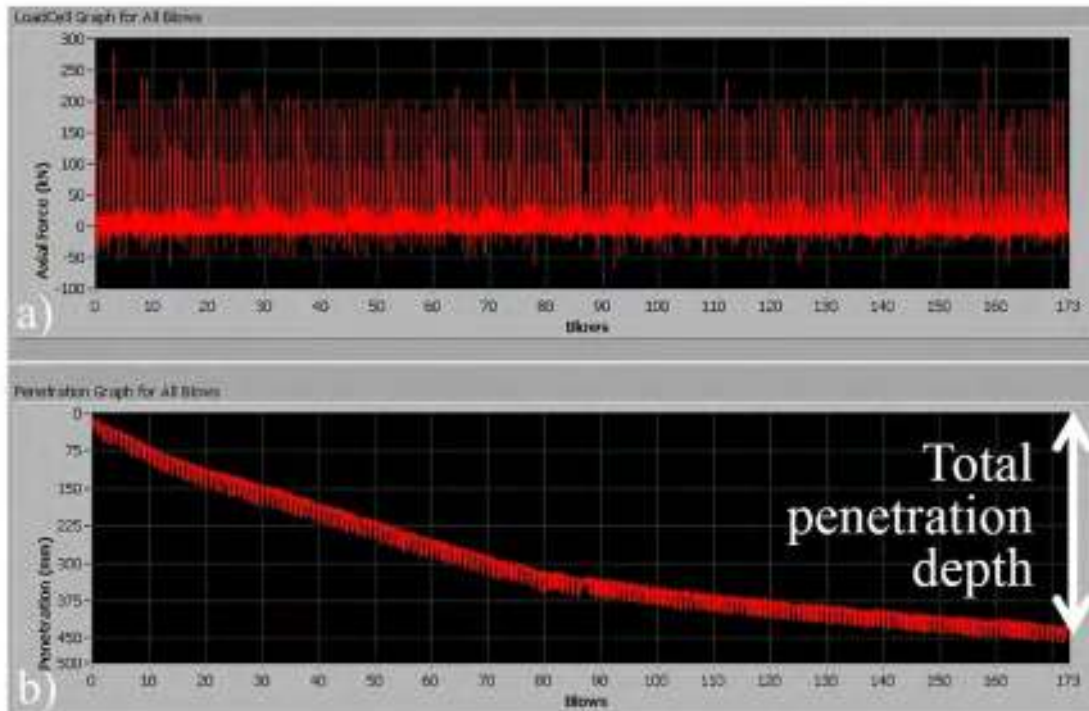


Plate 7: The automation of N-value measurement by SPT N-value Monitor at field

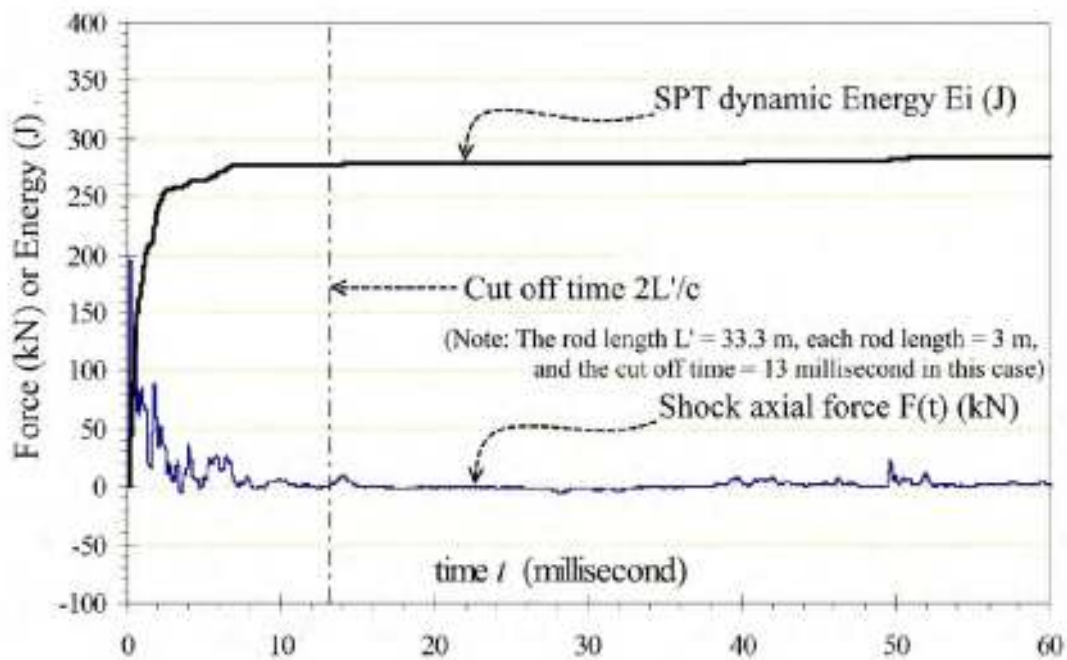


Figure 4: A typical result of the time variations of shock axial force and the calculated SPT dynamic energy due to one hammer blow at field

3.5 Energy Efficiency and Assessment

The integrated software can use the real time shock axial force data to calculate and assess the energy efficiency. The following equation of the force squared-time (F2) method is used to calculate the SPT dynamic energy E_i that is delivered to the drill rods by a SPT hammer blowing.

$$E_i = \frac{c}{AE} \int_0^{\Delta t} [F(t)]^2 dt \quad (1)$$

where, $c = \sqrt{E/\gamma}$, E , γ and A are, respectively, the Young's modulus, density and the cross-section area of the steel connector rod above and below the load cell, $F(t)$ is the measured shock axial compressive force in the connector rod as a function of time, and Δt is the duration for the dynamic energy transmission. An example is shown in Figure 4 above.

The SPT energy efficiency (ERr) is defined as the percentage ratio of E_i over the theoretical free fall energy released by the gravity potential energy of the hammer drop ($= 63.5 \text{ kg} \times 9.8 \text{ m/s}^2 \times 0.76 \text{ m} = 472.95 \text{ J}$). For our 106 monitored field SPT tests in Hong Kong, the ERr is between 32% and 80% for the total rod length (depth) between 3 m and 48 m.

3.6 Reliability, Durability and Demonstration

During the two years from June 2003 to May 2005, the following research activities were carried out. A SPT apparatus was established and used in the laboratory of HKU's Department of Civil Engineering. The prototype devices for automation of the SPT measurement monitoring and recording were designed, fabricated and calibrated in the laboratory. The new devices were testing and verified using the laboratory SPT apparatus. The number of the recorded hammer blows was 846 in laboratory. Field trials, tests and verifications of the new devices were carried out at actual SPT works at nine ground investigation sites in Hong Kong. At the nine sites, a total of 136 SPT tests were measured using both the conventional manual method and the newly invented digital system. The depths of the 136 SPTs were from 2 to 52 m. The total number of the recorded hammer blows was 10,799. The recorded SPT N-values are from 1 to more than 200. The prototype devices were improved and refined for many times. Evaluation and assessment of the monitored SPT results were carried out by comparing the automatic monitored results with manual measurement results. Monthly and quarterly reports were prepared to summarize the research activities and to report to Housing Authority. Monthly and quarterly progress meetings were undertaken with Housing Authority's representatives. Site demonstration of the new prototype devices to engineers in Hong Kong were carried out on June 29, 2005, as shown in Plate 8.

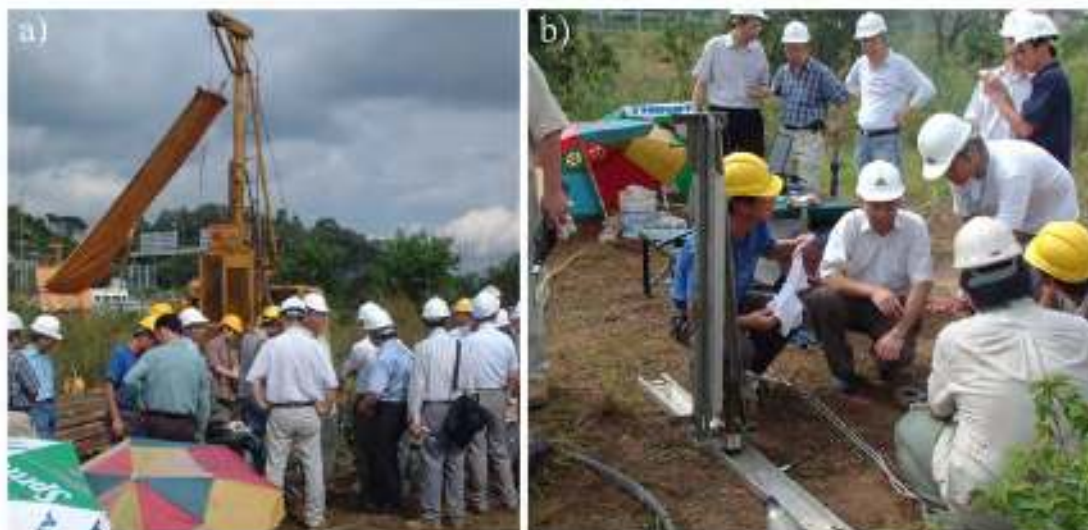


Plate 8: Site demonstration of the automatic SPT Monitor on June 29, 2005 at ground investigation site in Hong Kong

- 8a) Typical rotary drill rig deployed for the Automatic SPT Monitoring;
8b) Testing for the shock penetration transducer set

4 CONCLUSIONS

This paper addresses and attacks the manual measurement issue in quality of SPT works in Hong Kong by the development of the automatic SPT monitor. The monitor can automatically record and measure the entire testing process of the Standard Penetration Test in Hong Kong. It has a SPT Tip-depth monitor and a SPT N-value monitor. They are portable and can be non-destructively mounted onto existing SPT

apparatus. The real time data is traceable and independent. They can be used to enhance the quality of the SPT in ground investigation in Hong Kong and around the world. They can be used to further examine the hammer energy efficiency and effect of drill rod length.

ACKNOWLEDGEMENTS

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