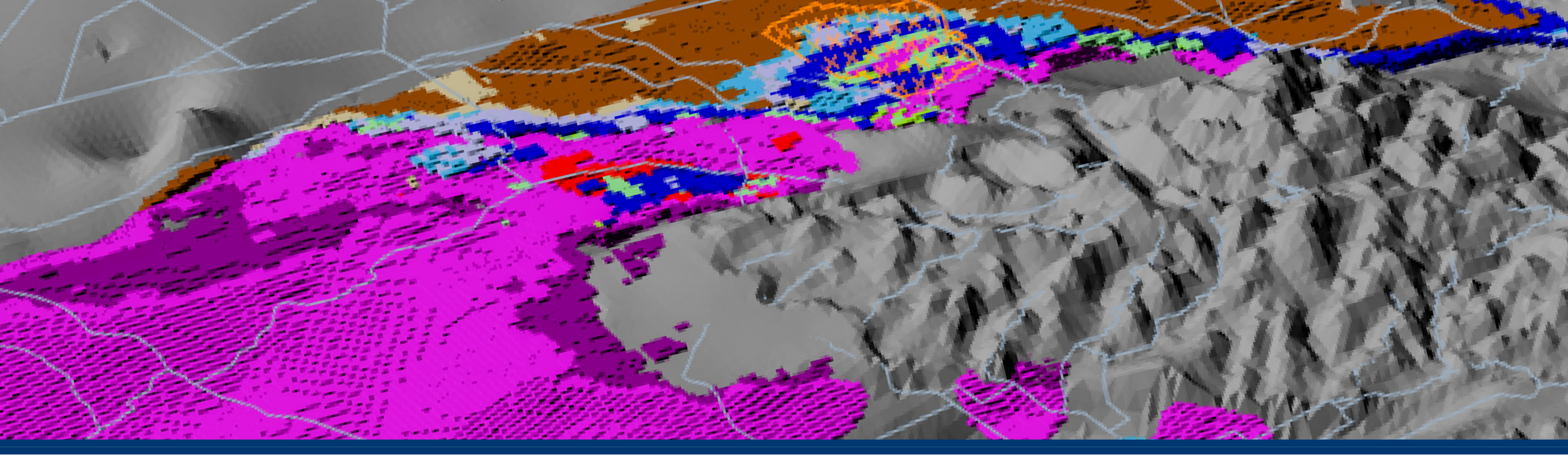


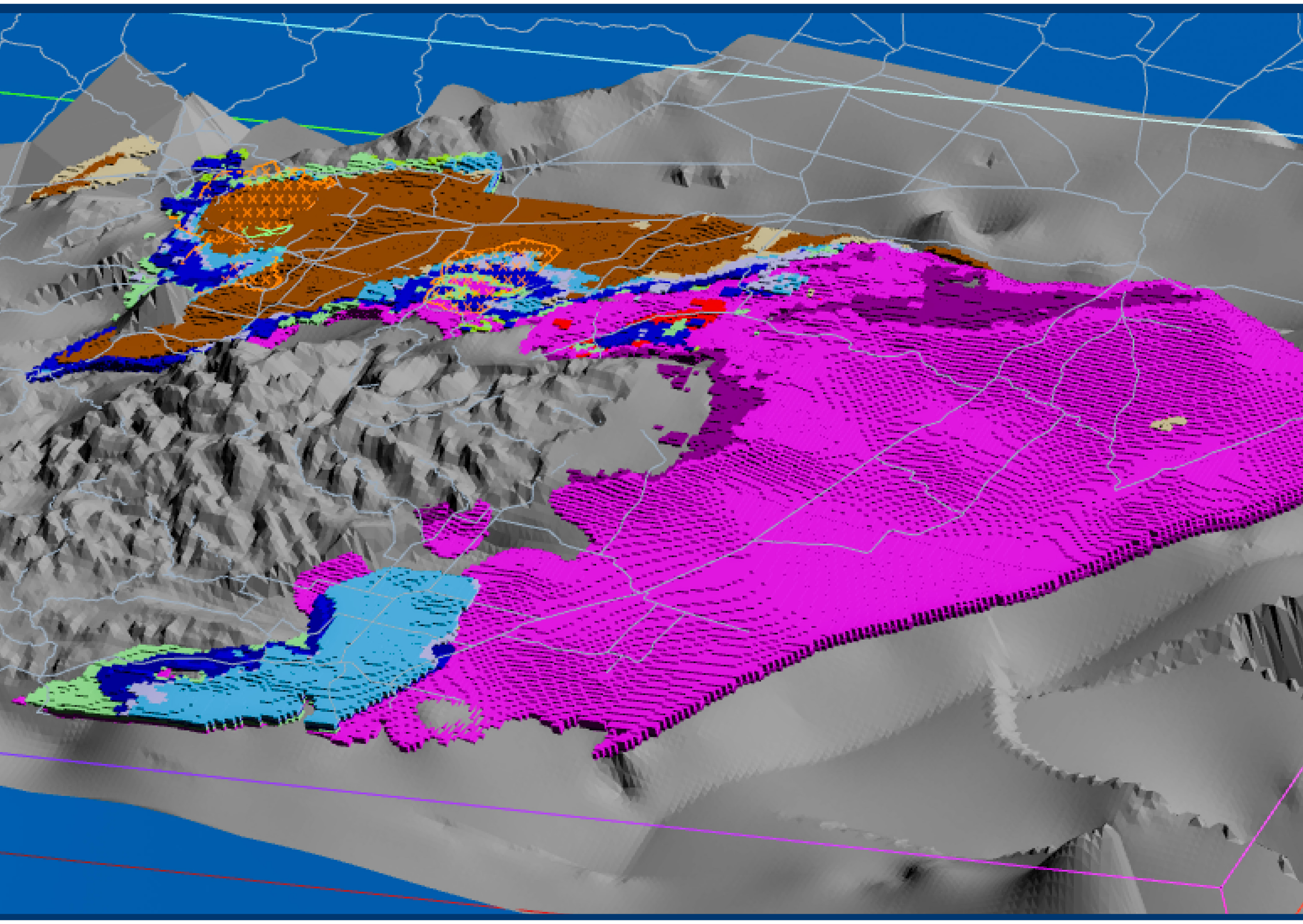
PROCEEDINGS

2022 AUSTRALIAN GEOMECHANICS SOCIETY VICTORIAN SYMPOSIUM

Digital Geotechnics

Wednesday 26 October 2022, 8:30am – 6:00pm

State Library Victoria, 328 Swanston Street Melbourne



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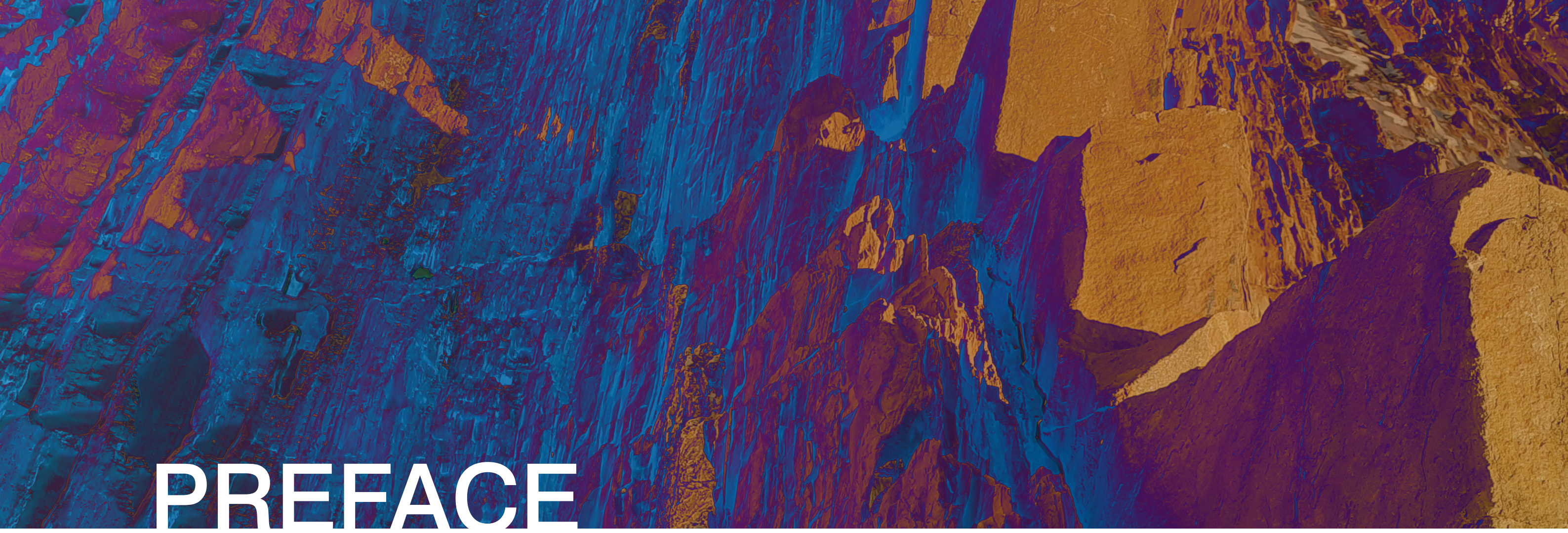

PREFACE

The Victorian Chapter of the Australian Geomechanics Society (AGS) is pleased to announce the 2022 AGS Victorian Symposium titled "Digital Geotechnics" to be held on 26 October 2022.

The symposium forms part of the continuing program of events organised by the Victorian Chapter of the AGS. The event aims to showcase recent developments in digital geotechnics and applications of digital geotechnics in practice. This symposium is to be held as a full-day face-to-face event in Melbourne.

The Architecture, Engineering and Construction (AEC) sector is undergoing a digital revolution with the array of digital technologies quickly gaining momentum within the civil space – but what role will the geo-profession play? Will it remain largely on the periphery or a pioneer in the adoption of digital data and digital technologies?

Digital engineering as a practice area is rapidly evolving and characterised by large quantities of unstructured data/information and many new and emerging technologies that may include; UAVs, photogrammetry and 3D scanning, sensing technologies, common data environments, BIM, CAD and GIS, digital twins, augmented and virtual reality (AR/VR), machine learning, artificial intelligence, parametric and algorithmic modelling and advanced simulations. Recognising the value to the engineer/geologist and successfully implementing these technologies within a ground engineering organisation is a significant challenge at this time.

Despite these challenges, new forms of digital data such as drone survey, LIDAR, InSAR and instrumentation data are becoming commonplace on geotechnical projects and being integrated with 'traditional' forms of geotechnical data such as borehole records, survey data, CPT logs etc. Those in the geoprofession, from practitioners to contractors, are increasingly recognising the importance of geodata capture, storage, visualisation, analysis and data sharing in the success of their ground engineering projects.

Integrated digital data on geotechnical projects have the potential to unlock improved geotechnical characterisation, increased collaboration, improved visual representation and sophistication in the ground model and advances in the efficiency of numerical simulation and the extent of the design space explored. Realising these benefits is leading many to recognise the importance of efficient and accurate digital geodata acquisition methods and the need for standardised approaches for data exchange. The days of the paper log are fast coming to an end.

This symposium seeks to provide the geo-profession with insight into the world of digital engineering and will bring together practitioners, researchers, contractors and subject matter experts to share and discuss their expertise on the topic of digital engineering and its application to geotechnical engineering and engineer geology. Best practices, case histories and innovative methods adopted in the field of digital engineering will be presented and discussed.

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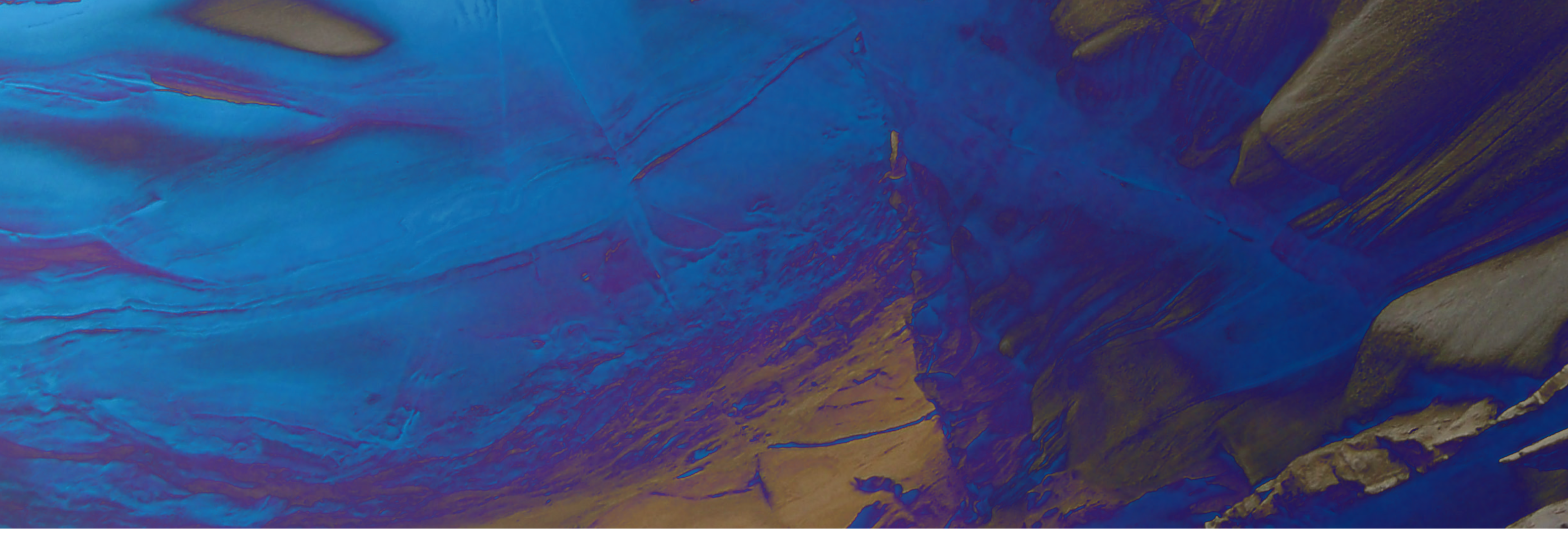

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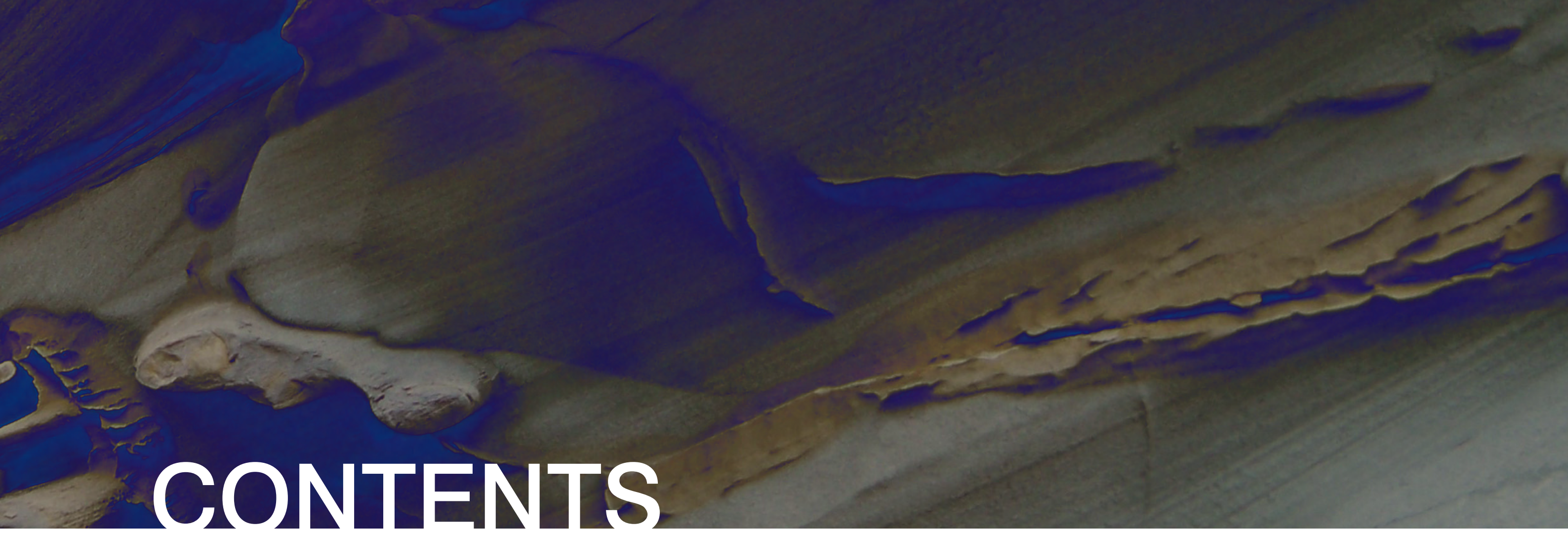
Cover image: Latrobe Valley 3D coal model coloured by coal seam, courtesy of Chris Osborne (Department of Jobs Precincts and Regions)



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ABOUT THE KEYNOTE SPEAKERS

CHRIS OSBORNE

Senior Geologist

Department of Jobs, Precincts and Regions

Chris Osborne is a geologist who specialises in 3D coal resource modelling. He is currently working on the Latrobe Valley Regional Rehabilitation Strategy for the Department of Jobs, Precincts and Regions.

Chris completed a Bachelor of Science (Honours) at Latrobe University in 1997.

From 1998, Chris worked as a field geologist for the Geological Survey of Victoria (GSV) where he was part of the geological mapping team responsible for producing geological maps and reports.

In 2012 he spent 18 months with a Mantle Mining helping to better define a coal resource near Bacchus Marsh as part of their drilling program. He then returned to the GSV where he was part of the "3D Victoria" team building a 3D, crustal scale geological model of the State.

In 2016 he moved across to Clean Coal Victoria, then to Low Emissions Resources, then to Coal Resources Victoria, still as part of the Department. His work here involved supporting the exploration/mining industry and the Department with specialist analysis of the coal resource model to inform decisions about the utilisation of the brown coal resource. He is currently using the same model and similar skills to inform decisions around the rehabilitation of the Latrobe Valley brown coal mines. Part of this modelling journey has forced him to (attempt to) understand data collection, storage and analysis solutions.

ERIC BUGEJA

Chairperson

BuildingSMART Australasia

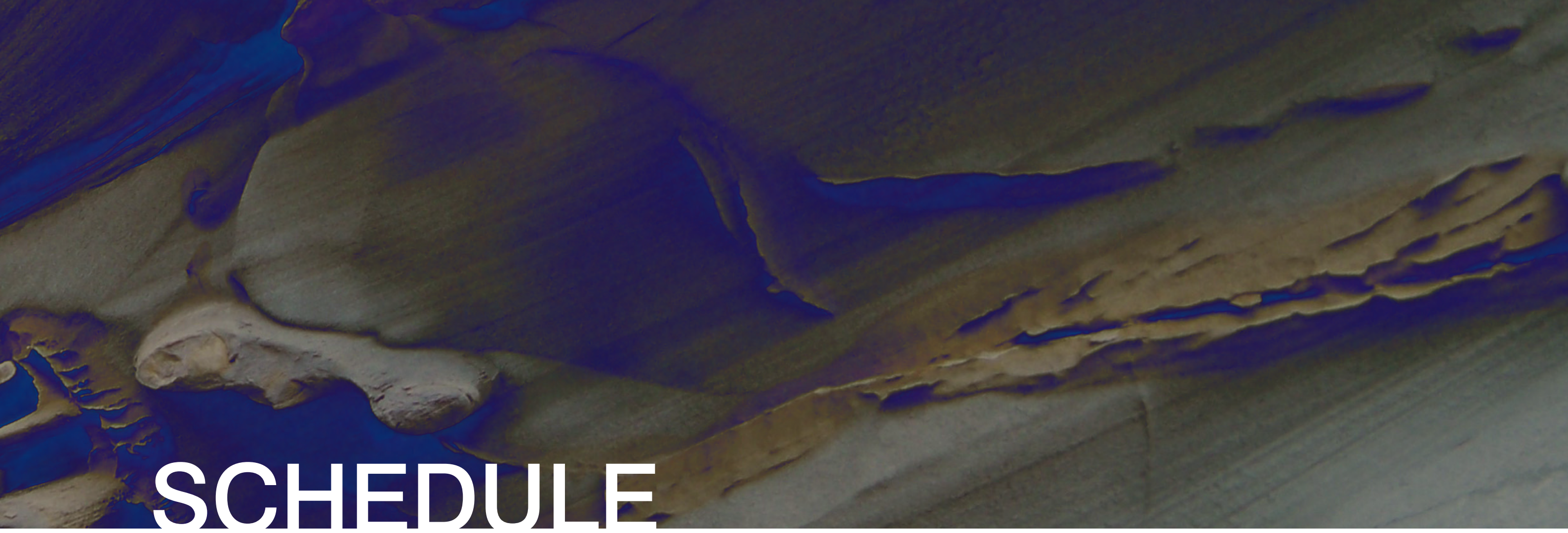
Eric Bugeja is the Chairperson of BuildingSMART Australasia and an Executive member of the Australasian BIM Advisory Board (ABAB), he has been an advocate for technological change in the AEC sector through his various roles, ranging from: Manufacturing Automation, Start-up advisor, Design manager on multidisciplinary projects, Project director on rail infrastructure projects, Group Manager of a multidisciplinary Architecture and engineering team, and Digital Integration for a Tier 1 Contractor. These roles have provided him with a unique perspective of the opportunities available through the application of technology across various sectors. Harnessing technology to promote collaboration, improve efficiency of design, speed up construction and improve Asset operation and maintenance.

YUXIA HU

Professor

University of Western Australia

Prof. Yuxia Hu obtained her PhD degree in 1991 in costal and offshore engineering. After graduation, she has been working in China, Japan and Australia. After she joined the Geomechanics group (later Centre for Offshore Foundation Systems – COFS) at the University of Western Australia (UWA) in 1992, she started to work with Prof. Mark Randolph to develop a practical approach to analyse soil-foundation interactions involving large deformations of soils - LDFE/RITSS (Large Deformation FE/Remeshing and Interpolation Technique with Small Strain model). The LDFE/RITSS method has been implemented in different commercial software and applied to many offshore soil-structure interaction problems successfully, such as pipeline, spudcan, anchors, skirted foundation, penetrometers and foundation for renewable energy offshore. Prof. Hu has published more than 100 journal and conference papers on the improvement of offshore foundation designs, and corresponding design guidelines. Her contribution to the geotechnical community has been recognised by 2005 IACMAG Junior/Senior Paper Award, 2006 BGA Best Paper Award and 2018 ICE Telford Premium Award. Her research also includes collaborations with local pavement engineers to work on sustainable designs in pavement to serve local geotechnical community. She has served as Head of Department of Civil, Environmental and Mining Engineering in 2018-2022.



SCHEDULE

8:30 am **Welcome and opening remarks**

SESSION 1
GEODATA ACQUISITION

8:40 am **KEYNOTE ADDRESS**
The 3D Digital Geological Model of the Latrobe Valley Coal Resource – the benefit of building versatile models
Chris Osborne (Department of Jobs Precincts and Regions)

9:20 am **Presentation**
Utilising laser scanner and unmanned aerial vehicle (UAV) geotechnical data capture to manage visitor safety in the Buchan Caves
Chris Coulson (AECOM)

9:35 am **Presentation**
Mapping expansive soils from space
Chris Fagan (Sixense)

9:50 am **Platinum Sponsor Presentation**
Chadwick Geotechnics

10:05 am **Q&A**

10:15 am **Morning Tea**

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DATA MANAGEMENT AND DIGITAL TWINS

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openBIM standards for geotechnic information exchange
Jon Mirtschin (Geometry Gym)

11:45 am **Presentation**
Implementation of the AGS geotechnical data transmission format: the Brazilian experience
Diego Gazolli Yanez (ATC Williams)

12:00 pm **Presentation**
Hoover Dike USA – Experiences with the use of a digital twin in specialist civil engineering
Jonas Gottwald (BAUER Spezialtiefbau GmbH)

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Global Synthetics

12:25 pm **Q&A**

12:35 pm **Lunch**

SESSION 3
ADVANCED NUMERICAL MODELLING

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Comparing numerical modelling finite element results with full scale instrumented pile response in weakly to moderately cemented soil
Bhavikh Riyat (Golder/WSP)

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Initiation of Internal Erosion in Earth Dams: A Particle-Scale Computational Approach
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14:45 pm **Gold Sponsor Presentation**
Geofabrics

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15:05 pm **Afternoon Tea**

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Digital optimisation workflow in early project phases and what it can bring when looking at the MacLeamy curve
Luan Nguyen (BAUER Spezialtiefbau GmbH)

16:10 pm **Presentation**
Digitalisation and Automation of Road Materials Compaction: SPARC Intelligent Compaction Kit
Arooran Sounthararajah (Monash University)

16:25 pm **Presentation**
Powering Pile Design through Parametrics
Elise Verrocchi and Nathan Ficatas (ARUP)

16:40 pm **Q&A**

16:50 pm **Closing Remarks and Thanks**

17:00 pm **Canapés and drinks**

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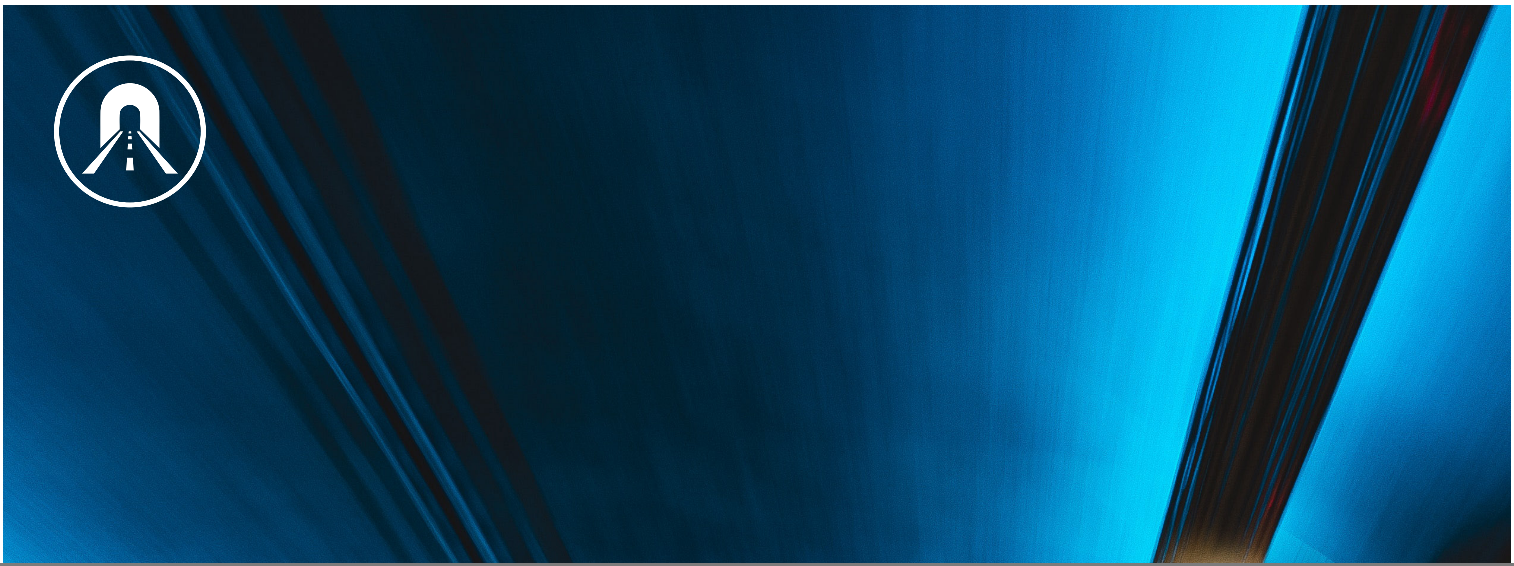

SESSION 1
GEODATA ACQUISITION

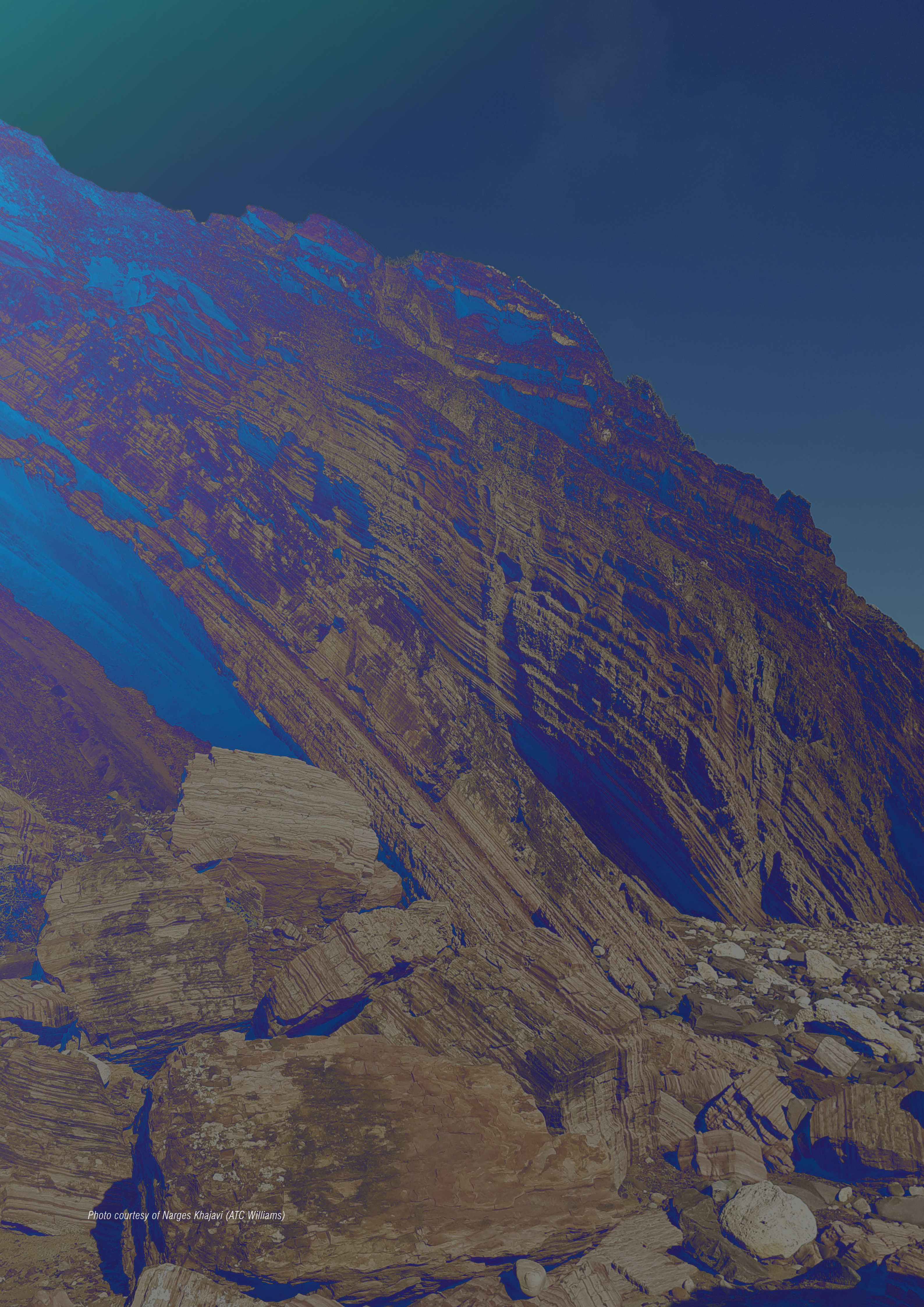

Photo courtesy of Narges Khajavi (ATC Williams)

The 3D Digital Geological Model of the Latrobe Valley Coal Resource – benefits of building versatile models

C. R. Osborne[1]

[1]Rehabilitation and Innovation, Earth Resources Policy and Programs, Forestry, Resources & Climate Change Department of Jobs, Precincts and Regions, Melbourne, Victoria 3000; M: 0448380372, email: christopher.osborne@ecodev.vic.gov.au

ABSTRACT

The 3D Digital Geological Model of the Latrobe Valley Coal Resource captures and safely archives 90 years of knowledge accumulated by the State Electricity Commission of Victoria and other workers that was previously accessible only as paper records. 9,086 bores have been modelled over a total area of 4,916 km^2, to include all onshore Gippsland Basin brown coal fields. Roofs and floors have been created for the sixteen thickest brown coal seams and splits off main parent seams. Seventeen coal quality parameters are incorporated into a block model.

The past few years have seen the purpose of the model changing. It is no longer a tool solely used to inform coal development opportunities, but is also used to inform coal mine rehabilitation. This paper highlights the relative strengths of explicit (data driven) versus implicit (significant amount of user input required) modelling based on the spatial coverage/resolution of the data. Also highlighted is the need for greater transparency in the strengths and uncertainties in 3D spatial models.

Keywords: Brown coal, resource model, Latrobe Valley, mine rehabilitation

1 INTRODUCTION

The 3D Digital Geological Model of the Latrobe Valley Coal Resource was developed for the State Government of Victoria in 2003 as a regional scale tool for supporting the management of the coal resource, matching coal to development opportunities and informing land use planning. Over time, the spatial extent of the model has been increased to cover more of the coal resource. This model helps stakeholders (Government, town planners, mining and exploration companies) make better informed decisions relating to coal resource development and future mine rehabilitation.

2 GEOLOGICAL SETTING

The 3D model covers the Latrobe Valley and Seaspray Depressions in the onshore part of the Gippsland Basin, which is one of the world's premier coal and petroleum basins (Figure 1). The Depressions are a late feature associated with Australia–Antarctic separation in the mid-Cretaceous (100 Ma). Rift fill sandstone of the Lower to Middle Cretaceous Strzelecki Group and Palaeozoic rock underlie and crop out around the Depressions. The South Gippsland highlands were initiated during shortening of these rocks in the Middle Cretaceous. Rifting that recommenced in the Late Cretaceous formed the Latrobe Valley and Seaspray Depressions.

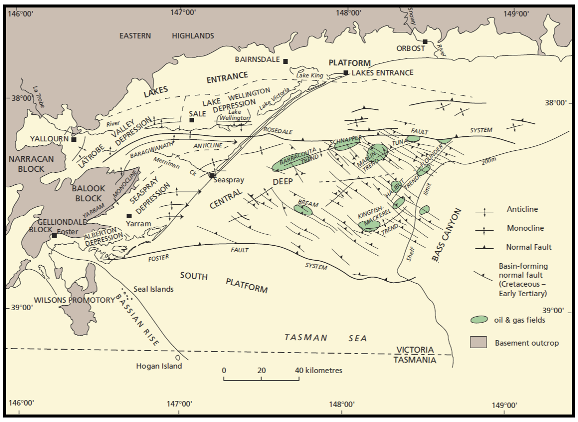

Figure 1. Gippsland Basin — tectonic setting and structure map. Also showing the main oil and gas fields offshore part of the basin. Modified from Abele et al. (1988).

Swamps that formed in this Depression were filled with organic material during the Oligocene to Miocene—the Traralgon, Morwell and Yallourn formations (Figure 2). The great thickness and uniformity of many of the coal seams over large areas is consistent with slow and steady rates of subsidence in the Depression.

Reactivated reverse faults in the underlying Strzelecki Group do not persist into the poorly consolidated Traralgon, Morwell and Yallourn formations but are expressed as monoclines and folds. Erosion of the Traralgon, Morwell and Yallourn formations was followed in the Plio–Pleistocene by deposition of a veneer of gravel, sand and clay—the Haunted Hill Formation (Gloe & Holdgate, 1991).

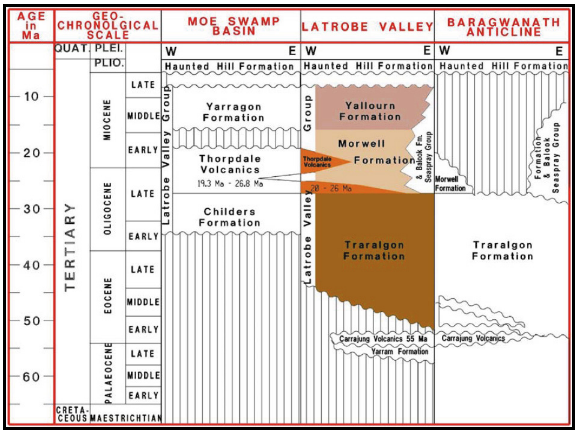

Figure 2. Stratigraphy of the Latrobe Valley Depression and neighbouring areas (after Abele et. al., 1988).

3 MODELLING

3.1 Input Data

The 3D model is based primarily on borehole/drillhole data (Figure 3) collected and compiled into the Latrobe Valley Coal Bore Database (LVCBD) as part of the systematic resource evaluation by the State Electricity Commission of Victoria (SECV) to support coal development.

To infill gaps in deeply buried Traralgon seams within the Seaspray Depression, selected deeper petroleum wells with fair control on coal seams were modelled along with coal bores.

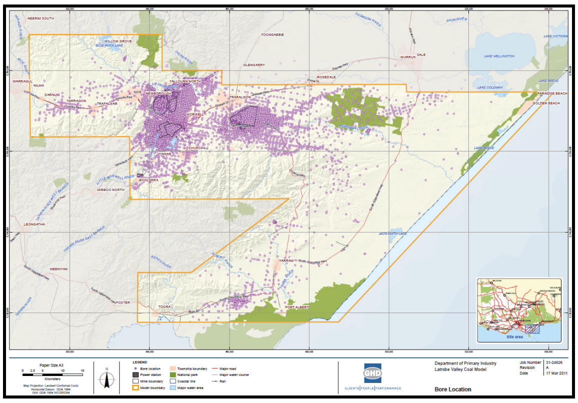

Figure 3. Borehole coverage of the model area

3.2 Methodology

The development of the coal model is undertaken using MineScape in two distinct steps.

i. A stratigraphic model of the geology developed; and
ii. Converting the stratigraphic model into a block model to define the distribution of coal attribute (quality) data.

The modelling methodology is described in detail by Jansen et al., 2003 and some of the key points are included here.

3.2.1 Stratigraphic Model

Roofs and floors have been generated for the sixteen thickest brown coal seams and splits (Figure 4 and Figure 5).

Formation	Seam / sub-seam / splits		Comment
Yallourn	Y	Y1	The splits Y+, Y3 and YP have been incorporated into the parent seam Y.
		Y2	
Morwell	M10		
	M1A	M1A1	The split M1A3 has been incorporated into the parent seam M1A.
		M1A2	
	M1B	M1B1	
		M1B2	
	M2	M2A	The M2C split (not shown) was not modelled. The split M2A1 has been incorporated into the parent split M2A.
		M2B	
Traralgon	TP		
	T1		The splits TRL and TRU have been incorporated into the parent seam T1.
	T2		

Figure 4. Modelled coal seams

The way that MineScape models these coal seams to produce the stratigraphic model is dictated by a set of "rules" which form a part of the schema. The schema controls all the modelling parameters that govern the way the structural model will be interpolated, such as stratigraphic relationship (conformable, nonconformable, transgressive…), seam splits, interpolation rules, limiting polygons, etc.

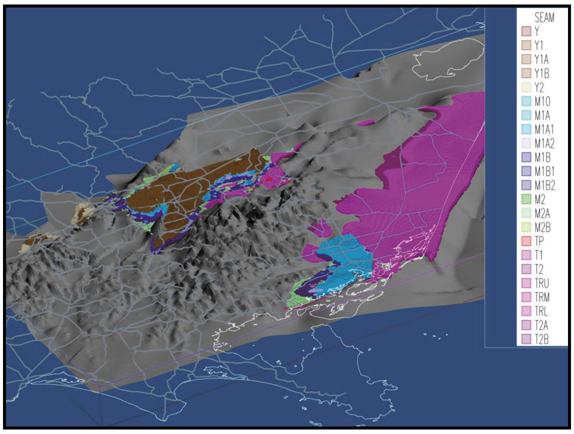

Figure 5. Latrobe Valley 3D coal model coloured by coal seam

3.2.1.1 Stratigraphic Model Validation

Roof and floor surface contours, and isopachs were generated for each coal seam and analysed visually. Anomalies in the data show up as bulls eyes and were reviewed and seaming updated where appropriate.

Modelled surfaces were also validated against hand drawn SECV isopachs for the Yallourn, Morwell 1B, Morwell M2 and Traralgon seams and splits. Ten regional cross sections were used to ensure consistency between the model and work of the SECV.

3.2.2 Block Model

Seventeen coal quality parameters have been incorporated into a block model. The block model was generated in MineScape using analysis from bores used in the stratigraphic model (Figure 6).

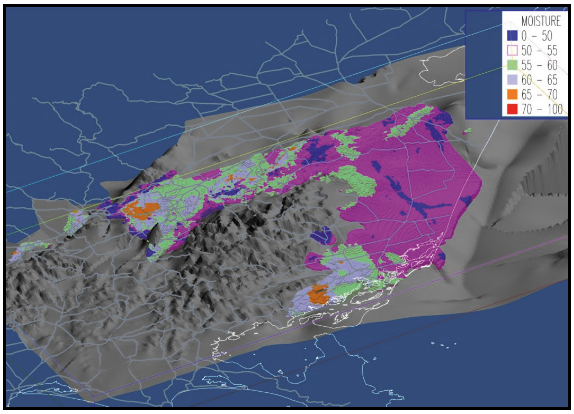

Figure 6. Latrobe Valley 3D coal model coloured by moisture (%)

3.2.2.1 Block Model Specification

The most basic block model parameter is the parent block size. This defines the maximum block size into which coal qualities will be interpolated. The parent cell size is 160 x 160 x 12 m (X, Y, Z). This size was selected to balance model resolution with performance. The 12 m vertical (Z) size was chosen to correspond to the maximum coal quality sample interval. The X and Y dimensions (160 m) were chosen to honour the borehole spacing and to balance model resolution with performance.

3.2.2.2 Block Model Interpolation Parameters

Three sets of interpolation parameters (Int_ash, Int_moist, Int_sodi) were used to interpolate coal qualities into blocks (Figure 7) and applied to qualities that behaved in similar ways. These are based on search distance (in metres), direction (defined by octants) and the inverse distance power.

Name	Search radii	Min octants	Min samples/oct	Max samples/oct	Inv dist power
Int_ash	5000	1	1	8	2
Int_moist	2500	3	1	8	1
Int_sodi	1500	3	1	8	2

Figure 7. Parameters used to interpolate coal qualities

3.3 Resource Calculation

As a validation process, the model was used to estimate coal resources and compared to previous resource estimates. It was found that there was good agreement between historical resource and modelled resource estimates. This is not surprising because both estimates employ the same methodology, with the MineScape model being a much more efficient means of estimation. Because resource estimates require average seam thicknesses from specific areas of interest, the model is capable of rapidly analysing hundreds or thousands of boreholes and their associated coal quality attributes.

3.4 Assumptions and Limitations

It is important to recognise that the confidence in modelled seams decreases with distance from bores. This includes confidence in seam presence or absence, seam roof, floor and thickness and seam quality. Seams need to be reviewed in the context of the bore data from which they are extrapolated. Bore densities across the model area range from metres to tens of kilometres. This is different for shallow and deep seams; with deeper seams only intersected by rarer deeper drilling.

The seam and quality interpolation parameters selected, to the extent that is possible, balance the need for resolution in areas with high bore densities (e.g., within mining tenements) and low bore densities (e.g., along the Ninety Mile Beach where Traralgon seams are deeply buried.

3.5 New uses for the model

The Latrobe Valley Coal model was originally created in 2003 and was last updated in 2011. Its original stated purpose was to "*to be used as a regional scale tool for supporting the management of the coal resource, matching coal to development opportunities and informing land use planning.*" While this fundamentally has not changed, the new "development opportunities" are constrained to fit within "*the future use of brown coal in a low emission context*" (Statement on Future Uses of Brown Coal, DJPR, 2017), and land use planning. This includes rehabilitation planning for the 3 coal mines in the Latrobe Valley involving:

- Simple calculations of pit volumes to determine water or material volumes
- Calculations of areas of exposed coal
- Overburden/lithology investigations
- Integration of LiDAR elevation data to analyse changes in landform
- Producing geological cross sections through the mine pits to inform batter stability modelling in other modelling software packages (Figure 8)

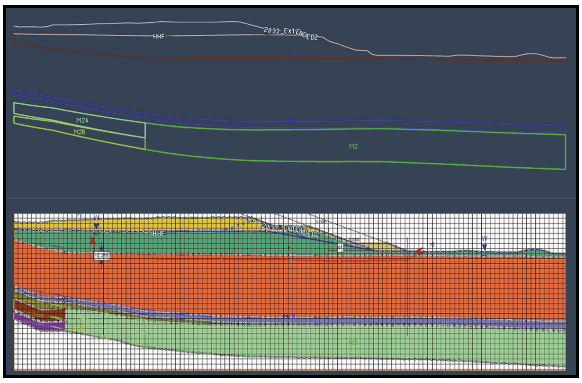

Figure 8. MineScape cross section (top image) through mine pit used as an input into geotechnical modelling software (bottom image)

3.6 Model Uncertainty

Although this is one of the best constrained 3D geological models of this scale, as for all models, there is still a degree of uncertainty as to its accuracy, especially in areas will low borehole density. It is important to consider where the uncertainty lies and how it is communicated and represented.

The data path can be used to examine sources of uncertainty:

- The logging of the bore: where to start and stop sampling to pick up the roof and floor of the coal seam has implications for the recorded seam thickness. The transition into the coal is often gradual.
- Assigning of names to coal seams: is this determined by the position in the drillhole, the lithotype, or the palynology (plant spores)? All these things require a subjective interpretation.
- Correlation and interpolation? between boreholes: how is interpolation managed, especially between areas of low and high borehole density
- The interpretation of the geophysical log to correct the lithological log: If you have ever had the pleasure of working with geophysicists or even radiographers, you will realise that interpretations of the same image can be quite different depending on knowledge and experience.

Then there are the modelling assumptions. There are 11 pages of rules and assumptions described in the original document that was released with the 2003 coal model (Jansen et al, 2003, Appendix VII). These include:

- The reinterpretation of lithology and stratigraphic codes to provide consistency across the model area and between the datasets associated with the 3 different mining licences.
- The “filling” of gaps in the coal seams within the bore logs. The software requires that no seaming gaps exist between the top and bottom of a seam, therefore, this gap filing is essential for the model’s operation.

Finally, there are the interpolation methods and grid resolution. Different coal qualities require different interpolations through the model space (Figure 7).

One way to do this in both two and three dimensions is to create buffers around data points which can be assigned a level of confidence (Figure 9). These might be based on:

- The distance to a borehole/between boreholes
- The distance to a borehole with coal quality analysis
- The distance to a borehole with downhole geophysics
- The distance to a borehole with palynology

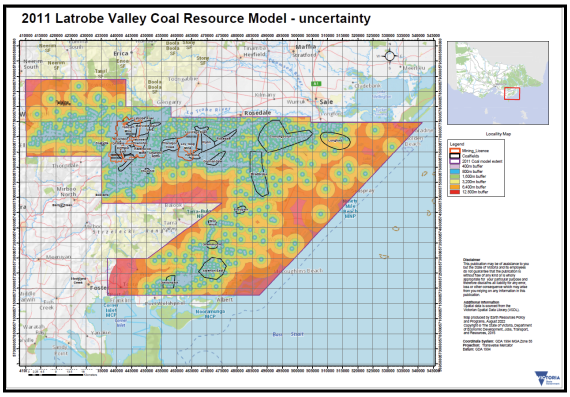

Figure 9. An example of a way model uncertainty could be illustrated

3.7 Model Outputs

The Latrobe Valley coal model is available as a “BASIC” package which gives an overview of the coal resource for the benefit of coal explorers, mine operators, proponents of new, low-emissions coal projects, the Government and land use planners that contains a metadata database, the borehole database and GIS layers for coal seam roofs, floors and isopachs. The other packages contain three-dimensional surface and block models of the coal resource in MineScape formats.

Outputs of the model are also available through GeoVic which is the Department’s free online mapping application. GeoVic enables the user to build earth resource related maps, perform searches and access data using a huge range of data sources. These model outputs include coal seam depths to tops and bases as contours, seam thickness isopachs, coal seam quality contours and coal seam stripping ratios, amongst others. The example in Figure 10 shows the stripping ratio (coal to overburden) for the Yallourn coal seam.

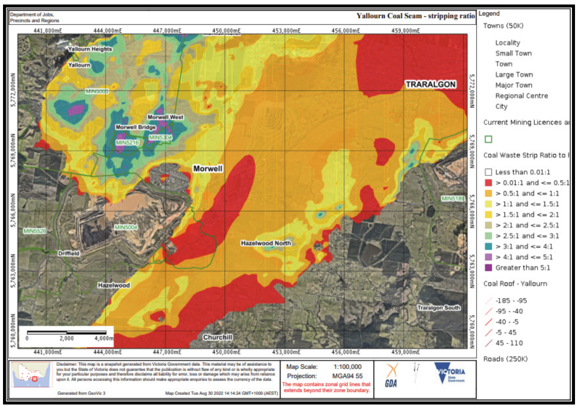

Figure 10. An example of an output from GeoVic

4 CONCLUSION

The Latrobe Valley Coal Resource model is a great example of how explicit and implicit modelling can be used to better define a resource and the challenges of finding the balance between the two approaches. The model was built from data collected over a 90 - year period. The data were recorded consistently and managed effectively, which has meant that a robust model can could be created. Although this model is so extremely well constrained by data (in places) it also highlights the need to be is clear that there is uncertainty in the model and understanding, quantifying and communicating this uncertainty is critical. This paper does not provide a solution, but encourages 3D modellers and model users to question the uncertainty of any model they are presented with, rather than accepting model results at face-value. Better outcomes are made possible by greater transparency in the strengths and uncertainties in 3D spatial models.

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Utilising laser scanner and unmanned aerial vehicle (UAV) geotechnical data capture to manage visitor safety in the Buchan Caves

C. Coulson[1] and J. Richards[2]

[1]Associate Director, AECOM, Collins Square Level 10, Tower Two, 727 Collins Street Melbourne, Vic 3008; PH (03) 9653 1234; email: chris.coulson@aecom.com
[2]Global Product Strategy Manager, Maptek, 63 Conyngham Street, Glenside, SA 5065; PH (08) 8338 9209; email: jason.richards@maptek.com.au

ABSTRACT

Some of the most impressive limestone cave formations in Victoria are found in the Buchan Caves Reserve. The caves are part of the Buchan–Murrindal cave system, located on Gunaikurnai Land and hosted in a large outcrop of Devonian-age cave and karst-forming limestones. The Reserve is one of East Gippsland's major tourism attractions, visited by large numbers of local, interstate and overseas tourists annually. Although the caves at Buchan are old (sediments sampled from within the Buchan Caves have been dated at greater than 750,000 years), ongoing periodic rockfalls into the caves, part of a natural process known as 'breakdown', are anticipated. The likelihood of breakdown events within engineering timescales poses potential risks to the visitor caves. A Maptek SR3 underground laser scanner and above-ground UAV photogrammetry have been used in combination to capture a 3D spatial relationship between the cave system and ground surface terrain. The resultant digital model assists in visualising this relationship, allowing for accurate identification and mapping of locations where breakdown hazards may be more likely to occur. The model also provides baseline data for future scanning that can help identify and monitor ongoing ground deformation. A geotechnical risk framework has been developed to assist Parks Victoria understand and manage the relative risks posed by cave breakdown, providing a formalised approach to managing safety in the Buchan Caves.

Keywords: geotechnical risk, limestone caves, laser scanner, UAV photogrammetry, pointclouds

1 INTRODUCTION

The Buchan Caves Reserve covers an area of over 100 Ha on Gunaikurnai Land in Buchan, East Gippsland. Buchan is a rural town approximately 350 km east of Melbourne in Victoria, Australia (Figure 1).

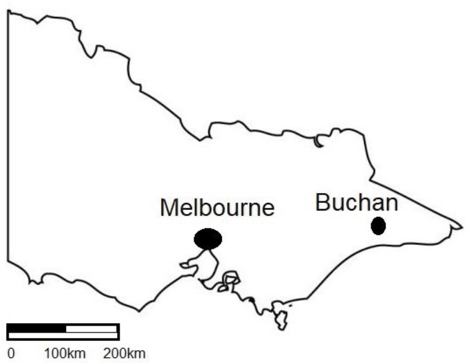

Figure 1. Location of Buchan, Victoria

The publicly accessible visitor caves (Fairy Cave, Royal Cave and Federal Cave) are part of the Buchan–Murrindal cave system, located in a large outcrop of Devonian-age limestones.

Parks Victoria manages the Reserve and caves, the first of which was opened to the public in 1907. The caves remain one of East Gippsland's major tourism destinations and attract large numbers of local, interstate and international tourists annually. Most of the natural caverns and passages have no engineered rock support, instead relying on the natural arching effect of the host rock to maintain stability. There are currently no formal assessment criteria or operations plan for identifying and managing geotechnical risk in the visitor caves.

2 GEOLOGY AND GEOMORPHOLOGY

2.1 Regional setting

The Buchan–Murrindal cave system is located within the Buchan Rift, a broad extensional trough containing a thick sequence of volcanic and clastic rocks and overlain by shallow marine limestones (Orth et al., 1995). The Buchan Rift forms part of the Lachlan Fold Belt, a sequence of Palaeozoic-age rocks that extends across most of Victoria, New South Wales, and Tasmania. The Buchan Caves Limestone, host geological unit of the visitor caves, was deposited about 380 million years ago.

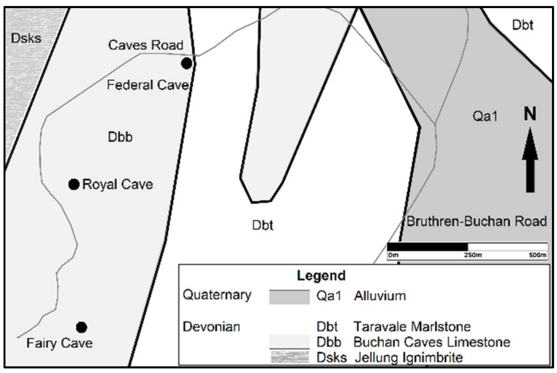

Figure 2. Site surface geology

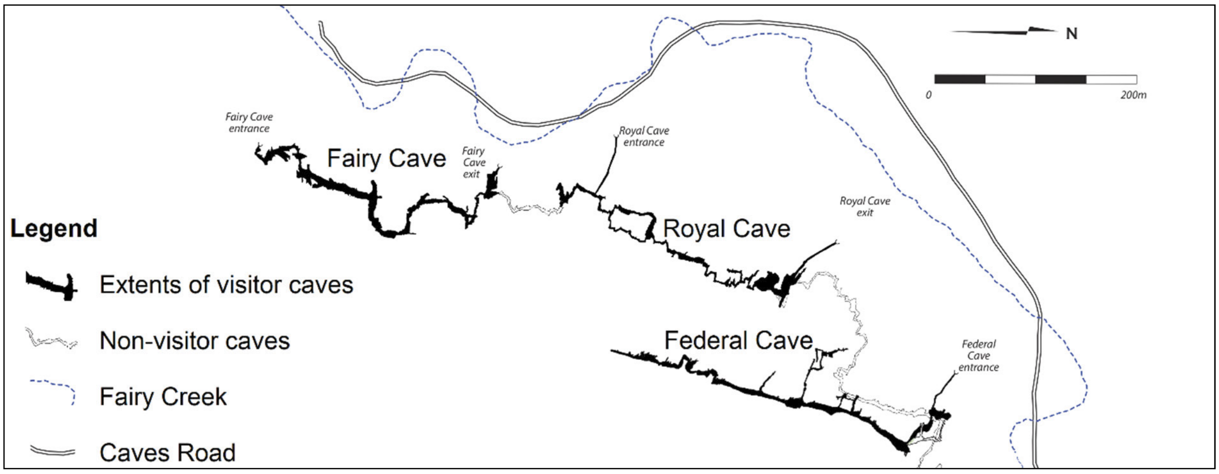

Figure 3. Plan showing Fairy Cave, Royal Cave and Federal Cave system

The depositional sequence was folded and lifted prior to volcanic eruptions 40 Ma that disrupted river systems and contributed to the erosion of deep valleys and cave development (Webb et al., 1991).

The surface geology of the Buchan Caves Reserve is presented in Figure 2 (adapted from Welch et al., 2011).

2.2 Cave formation and development

The Fairy Cave, Royal Cave and Federal Cave form part of a system of stream passage caverns (Figure 3). The main halls within these visitor caves are oriented north-south, parallel with regional strike. Shorter east-west passages connect the main halls.

Sediments and speleothems in the caves have been dated using various techniques, indicating cave formation occurred more than 750,000 years ago (Webb et al., 2003). The caverns generally formed with a flat roof, cut across bedding, following enlargement by historical karst forming processes beneath the groundwater table (Webb et al.,1992). A flat roof is still visible in many caverns (Figure 4). Regional groundwater is now below the level of the visitor caves, with stream flows continuing in deeper caverns.

2.3 Cave breakdown

Cave ‘breakdown’ refers to ‘the process by which cave ceilings, walls and floors fail, including slab failure, block failure, spalling and heaving, and the rubble produced by these processes’ (Osborne, 2002). Other authors provide similar definitions (Webb et al., 1992; Waltham 2002; Klimchouk 2005).

Figure 4. Cavern with a flat roof in Royal Cave

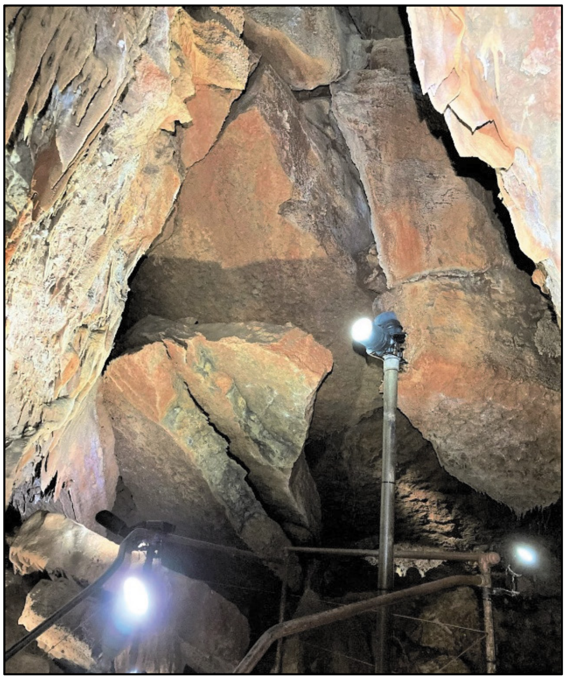

Figure 5. Example of breakdown and roof stoping

The visitor caves are exposed to air, water, and variations in humidity and temperature, and Osborne (2002) noted these factors contribute to the ongoing breakdown of the fractured Buchan Limestone. Meteoric water contributes to ongoing limestone dissolution. It also erodes clay and crushed fines from rock fractures or discontinuities as it percolates from ground surface, promoting their separation from the rock mass. Clay (where present) can also desiccate and become friable when exposed to dry or warm air, contributing to the unravelling of soil and rock around caverns.

Once initiated, cave breakdown continues as a progressive failure of individual beds or blocks that advances upwards, in a process known as 'roof stoping' or 'cavity migration' (Waltham, 2002) (Figure 5). Osborne (2002) notes that the breakdown blocks are often bound by pre-existing discontinuities in the limestone that have been opened and weakened by weathering, rather than the fracturing of intact rock. Breakdown continues until a sufficiently thick bed can support the roof, a stable arched zone of compression can form, or roof migration extends to ground surface, resulting in a collapsed doline (Osborne, 2002).

Roof breakdown results in the accumulation of soil and rock debris on the cavern floor, with size and form of breakdown piles being influenced by the processes acting in the caves. Breakdown clast sizes observed in the Buchan Caves range from fine soils to large boulders. The finer-grain material tends to accumulate on cavern floors, while in many places larger boulders remain wedged in cavities and fissures. Such boulders are supported on cavern sidewalls, and often on other boulders. Stream flows can erode finer material to leave clast-supported cobbles and boulders in the breakdown piles, while calcite precipitation and speleothem development can cement and stabilise breakdown material. Osborne (2002) also notes that breakdown piles continue to fragment over time, through processes such as crystal wedging, dissolution from acid released from pyrite (when present in the host rock), and from dissolution when submerged below the water table. This removal of breakdown material results in continued enlarging of the cave void.

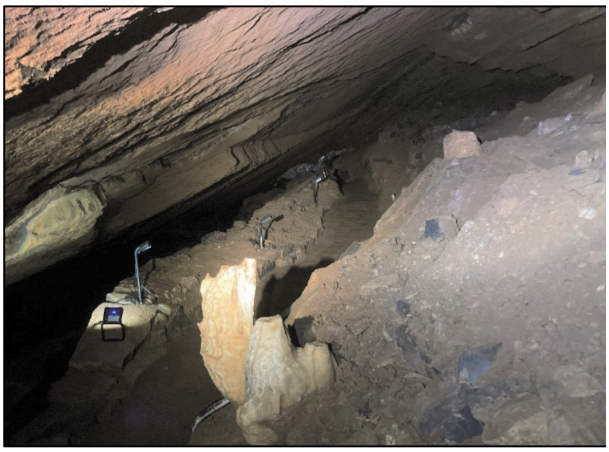

Figure 6. Bedding units in Federal Cave roof and corresponding breakdown pile on floor

Figure 6 shows thick inclined bedding units forming the roof of the Federal Cave. Rock and soil debris, from the breakdown of the original cavern roof, has accumulated as piles lining the cavern floor. Breakdown of cavern roofs, sidewalls, and of debris piles within the caves could pose potential hazards to the management of underground infrastructure and safety of cave visitors and Parks Victoria staff.

2.4 Condition of visitor caves

Park records from the early 20th century describe significant work to remove the products of roof breakdown, and to improve access prior to opening the caves to visitors. Drilling-and-blasting was used to enlarge smaller natural passages, and to assist with the removal of large boulders. Loose rock and other breakdown material were scaled back and removed from the caves to create access paths. The Victorian Mines Department recommended a range of safety improvements to the Caves Committee of Management between 1950 and 1980. These included scaling back potentially loose rock where observed, and the installation of a small number of rock bolts in the Princess Royal Chamber of the Royal Cave. It is understood these, and other similar recommendations, were progressively implemented by the Committee of Management and later Parks Victoria.

There are no recorded rockfalls or instances of natural cavern instability in the period since the caves were opened to the public approximately 100 years ago.

3 RISK-BASED APPROACH TO GEOTECHNCIAL MANAGEMENT OF THE CAVES

Parks Victoria Risk Management Guidelines assist staff to undertake corporation-wide risk assessments, and assist decision-making processes within the organisation (Parks Victoria, 2015). The Guidelines follow the common approach of identifying hazards, assigning frequency and consequence descriptors, and using a risk matrix to determine a 'level of risk'. Being a corporate-wide guideline, the 'consequence' descriptions are broad and 'likelihood' timeframes short when considering geological risks.

The Australian Geomechanics Society Practice Note Guidelines for Landslide Risk Management (AGS, 2007) contains geotechnically focussed qualitative 'consequences' and 'likelihood' timeframes that span engineering design-life and geological timescales. The 'risk implications' in AGS (2007) were developed to assist managing authorities to action mitigations.

Elements of the Parks Victoria Risk Management Guidelines and Australian Geomechanics Practice Note Guidelines for Landslide Risk Management have been adapted to develop a qualitative risk assessment for roof breakdown hazards within the Buchan Cave System. Combining these approaches provides a framework consistent with risk management practices within Parks Victoria.

3.1 Potential breakdown hazards

The first step in adopting a risk-based approach is to identify hazards. The proximity of a cave to ground surface, and the presence and condition of breakdown, are important factors that influence the likelihood and potential frequency of future rockfalls. Observations of these factors formed the basis of cave inspections and the subsequent risk assessment.

Other factors, including natural variability in geological structure and material properties, and human-induced modifications to the caves will also influence the likelihood of rockfalls. Such factors need to be considered when high-risk locations are identified, and specific breakdown hazards require management.

The approach outlined in the following sections was adopted when assessing the relative likelihood of future breakdown events.

3.1.1 Roof thickness to cave width ratio

Waltham (2005) suggests a cave roof in karstic limestone is stable under engineering loading when the ratio of rock roof thickness to cave width is 0.7 or greater. Locations with a ratio of 0.7 or less are considered to have a greater likelihood of a future breakdown event compared with a ratio greater than 0.7, particularly if the ground above the caves carries engineering loading.

3.1.2 Rock mass stress relief

The in situ principal stresses in a rock mass are distributed around an opening such as a cave, producing regions of increased compression and tension. The orientation and magnitude of stress redistribution depends on factors such as the shape of the cave and the ratio of in situ horizontal to vertical stress (Hoek, 2006). Reductions in compressive stress in the crown of a cave can result in the loss of rock-arch stability, promoting cavern breakdown. Ongoing erosion at ground surface and rock mass weathering processes will continue to influence the distribution of stresses above the roof of shallow caves.

Cave cross section dimensions and the thickness of the rock cover assist in identifying locations that may have a greater likelihood of breakdown due to a reduction in compressive stresses above the crown. Locations where shallow caves are within a distance equivalent to three cave diameters have been identified as having a greater likelihood of reduced compressive stresses in the crown when compared to deeper caves. The proximity of other caves can also influence stress distribution around the visitor caves.

3.1.3 Presence and condition of breakdown

Chemical weathering, erosion, and water flow along defects have a greater potential to dislodge loosened soil and disarticulated rock blocks in locations where breakdown has commenced and is active. The likelihood of rock falls will be greater in areas where breakdown is present, and in an active/ progressive phase.

3.1.4 Path proximity to breakdown debris

Cavern breakdown has produced a range of debris in the caves, ranging from soil and cobble-size material to large boulders accumulating beneath, adjacent to, and above the access paths. Active erosion processes within the caves means breakdown debris has the potential to continue moving. Where the access path is close to breakdown debris, the likelihood of future movement presents a hazard to the path and visitors.

3.2 Hazard size and frequency

A lack of historic rockfall data means that quantitative assessments of hazard size and frequency cannot be estimated using traditional methods regularly adopted when investigating landslide recurrences.

Given the absence of specific descriptors applicable to a cave breakdown environment, the qualitive frequency descriptors in AGS (2007) were adapted and used (Almost Certain, Likely, Possible, Unlikely, Rare, Barely Credible). Factors, including the age of the caves, past karstic processes differing from present erosional processes, hazard removal and stabilisation efforts during the twentieth century, and the absence of recorded rockfalls were all considered when allocating qualitative descriptors.

3.3 Consequence of a hazard occurring

Qualitative descriptors were also adapted to define a range of credible and conceivable consequences in lieu of having no recorded rockfalls from the visitor caves. The Consequence descriptors (Catastrophic, Major, Medium, Minor and Insignificant) are consistent with those suggested for landslide hazards by the AGS (2007). Each description included an estimate of the size of the impact on the cave and surrounding area, and commentary regarding the likely scale of remediation required to stabilise and reopen the caves should a rockfall occur.

3.4 Risk assessment and management

The AGS Landslide Risk Management risk assessment matrix (AGS, 2007) was adapted to assign risks to the hazards identified in the visitor caves, with assessed risk, implications and actions incorporated into Parks Victoria works plan for ongoing management of the Buchan Caves.

4 DATA ACQUISITION TECHNIQUES TO SUPPORT HAZARD IDENTIFICATION

Detailed and accurate measurements of cavern width, depth below surface, and shape were required to complement visual geological observations to better understand relationships between physical dimensions of the caves, rock cover, and the potential for further breakdown across engineering timescales. To understand the spatial geometry of the cavern system in relation to the natural topography and known geological structures, 3D laser scanning was undertaken. This process to generate a 3D point cloud involved utilising 2 tripod-mounted Maptek SR3 laser scanners to scan the cave system. It was crucial that adequate overlap of pointcloud data was achieved between scanning locations to produce a continuous model. This allowed detailed data analysis, identification, and safe recording of key geological and geotechnical features, especially in areas that cannot be safely accessed. Maptek SR3 laser scanners were nominated on this project due to their high resolution and accuracy, supplemented with photo realistic imagery capture with lighting affixed to the scanners (Figure 7).

Figure 7. Maptek SR3 scanner with light

A DJI Phantom 4 Pro V2 unmanned aerial vehicle (UAV) was used to capture photogrammetry imagery of the topographical surface over the caves. This imagery was processed using the 3DFLOW 3DF Zephyr software to produce a spatially correct topographical surface.

Maptek PointStudio software combined the laser scanner datasets with georeferenced surface photogrammetry to create a 3D georeferenced dataset. This was subsequently used to assess distances between the ground surface and caves, identifying potentially unsafe or high-risk areas that required further attention (Figure 8).

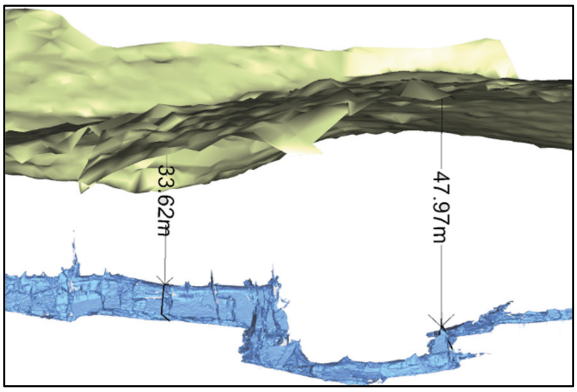

Figure 8. Distance between ground and caves

A spatially correct and georeferenced 3D pointcloud of the cavern system and topography also provides a visualisation tool that can be shared with the client and non-geotechnical applications (Figure 9). Other significant benefits of combining underground scanning with surface photogrammetry include:

- Rapid higher accuracy data acquisition compared to conventional surveying techniques
- Improved safety during data acquisition
- Collection of orientation and persistence of bedding planes and other discontinuities, allowing for detailed analysis and ground support design as required
- Removal of human bias when mapping geological data
- Recording the size and geometry of cave breakdown
- Provision of baseline readings for future scans

5 INVESTIGATION OUTCOMES

5.1 Roof thickness to cave width ratio and rock mass stress relief

The consolidated pointcloud dataset allowed measurements of cave width and cover to the ground surface to be easily extracted. The principal north-south oriented cave chambers are generally at depths ranging between 10m and 65m below ground surface. The ground cover reduces at the Royal and Federal Cave access adits, and the Fairy Cave entrance chambers. The measured cave width typically ranged from less than 1m to greater than 15m, although full span width is often difficult to measure where breakdown debris obscures the sidewalls.

The UAV photogrammetry imagery indicates rock outcrops over large areas of the slopes above the caves. A soil depth of 1m over rock was assumed when calculating the ratio of rock cover thickness to cave width, based on observations made on site. This assumption will be tested when specific hazards within the caves are being assessed further. Locations where the rock cover thickness to width ratio of 0.7 or less have been assigned as areas having a higher likelihood of breakdown. Locations where the caverns are within a distance equivalent to three cave diameters of the ground surface have been assigned an intermediate likelihood, while caves deeper than this nominally have lower likelihood. Breakdown conditions and local features within each cavern also influence the likelihood assessment. The proximity of non-visitor caves to the Fairy, Royal and Federal Caves, and the effects these have on ground stresses have not yet been considered, but provide an opportunity for future assessments.

5.2 Condition of breakdown

Visual observations and the digital pointcloud dataset were used to identify features associated with existing cave breakdown, including the presence of exposed bedding units in the roof and breakdown debris piles in caverns. The pointcloud data is particularly useful when determining dimensions and relative proximity of breakdown features to public access areas within the cave systems.

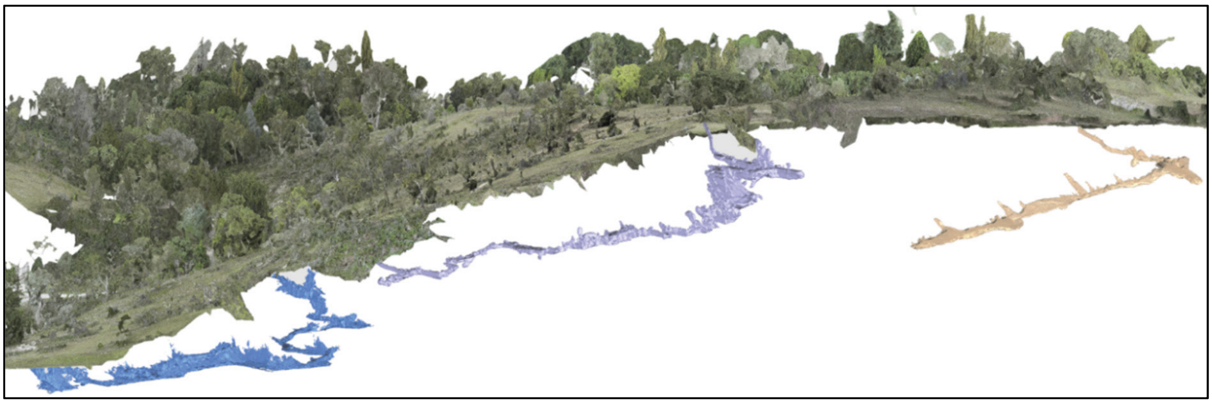

Figure 9. Merged UAV and SR3 Laser Scanner datasets showing surface topography and caves

The data has been used to estimate the size, volume and mass of boulders suspended over the public access path, which can then be used as the basis for quantifying hazards and nominating remedial engineering design and stabilisation works. The scanning enabled rock mass measurements in locations where breakdown has exposed bedding units and other defects. This data can be used as an input for the analysis and geotechnical design of ground support as required. Comparing the results of future scans with the baseline obtained during this study may help identify and quantify movement and erosion of existing breakdown, and potentially in areas where breakdown has yet to commence. Such movement might not be easily perceived by visual inspections but may indicate locations of future large-scale movement.

6 CONCLUSIONS

This paper presents the basis of a risk-based assessment criteria and framework to identify, assess and manage geotechnical hazards within the Buchan visitor caves. The combination of in-cave laser scanning and above ground UAV photogrammetry provides a rapid and repeatable way of developing an accurate three-dimensional digital model of natural underground caves and how the caves relate to the surrounding topography. Supplemented with visual geological assessments, an understanding of what cave-forming processes have occurred that are no longer active, and what processes continue to influence cave erosion, the digital model provides a tool that helps manage geotechnical risks associated with limestone caves.

The risk assessment documented in this paper is qualitative, due to the limited records of past cave instabilities. Comparing the results of future scans with the baseline obtained during this study may help identify ground movement not easily perceived by visual inspections, improving the dataset and allowing more detailed risk analysis to take place.

7 ACKNOWLEDGEMENTS

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Mapping expansive soils from space

J. Sala[1], A. Vradi[1], D. Albiol[1], M. Camafort[1], N. Devanthéry[1], J. Lluch[1], E. Audigé[2], C. Fagan[2], and R. Roberts[3]

[1]Sixense Iberia, 08907, Hospitalet de Llobregat, Spain; email: joan.sala@sixense-group.com
[2]Sixense Oceania, 5X92+77 South Melbourne, Victoria, Australia; email: eric.audige@sixense-group.com
[3]Auckland Council, Engineering Resilience, 5Q26+2X Auckland, New Zealand; email: ross.roberts@aucklandcouncil.govt.nz

ABSTRACT

This paper describes a new approach to detecting motion related to soil expansivity from ground motion time series data. Interferometric Synthetic Aperture Radar (InSAR) is a remote sensing technique used to detect and monitor ground and surface infrastructure motion with millimetric precision using SAR satellite imagery.

A total of 74 high resolution TerraSAR-X SAR images were used to produce ground motion time series covering a large urban area (450 km^2) in Auckland NZ between August 2019 and May 2022. Sixense's Atlas InSAR processing chain produced 7.6 million ground motion time series. Data science techniques were used to locate motion related to expansive clays swelling mechanisms.

Climate data from NIWA weather stations and soil data layers from Landcare Research NZ have been included to complement the analysis. 450k building footprints from Land Information New Zealand (LINZ) have been used to map those buildings potentially affected by expansive clay seasonal movements. Finally, ground instrumentation data from the Mount Eden City Rail Link (CRL) station construction site is used to validate the observed motion precision of Sixense's Atlas InSAR.

The end goal of this study is to provide the Auckland city council cost effective added value information in the context of climate change adaptation, where erratic weather patterns pose a serious risk to building foundations.

Keywords: InSAR, expansive soils, climate change, building foundations, assets at risk, structural health monitoring

1 INTRODUCTION

With excess precipitation and soil moisture increase, expansive clays generate uplift forces (Vorwerk et al., 2015). When this phenomenon develops at foundation level, it can lead to serious building damage due to differential settlement. Swelling clays can control the behaviour of virtually any type of soil, and climate change irregularities have enforced their impact.

This paper shows how InSAR ground motion time series is used in conjunction with climate observations to provide meaningful and actionable information for the city of Auckland, New Zealand. Radar satellite interferometry is a widely used technique to monitor terrain deformation over wide areas (Bamler et al., 1998). In the third chapter, the methodology chosen to filter out the motion related to expansive soil seasonality is presented together with the input datasets used.

Finally, we demonstrate how this massive amount of geoinformation is presented to the Auckland city council via Sixense's Beyond Monitoring GIS platform.

2 INPUT DATASETS

2.1 Atlas InSAR

The Auckland Atlas InSAR high-resolution ground motion time series amounts to a total of 7.6 million measurement points with 74 dates between August 2019 and May 2022. The density of points resulted in 40,000 PS/km^2 strictly in the urban area concentrated in man-made structures like buildings known for their phase stability along the period of study (Ferretti et al., 2000). This study period enables the identification of seasonal patterns since it contains more than a full cycle (winter/summer – dry/wet season).

TerraSAR-X acquisitions in StripMap mode have a ground pixel resolution of 3x3 metres and the image footprint is approximately 50x30 kilometres on the ground. The angle of incidence (α) of the acquired images over Auckland is 33 degrees. Figure 1 summarizes the acquisition coverage and geometry of the source imagery. Given the incidence angle of 33 degrees, the potential variation of the observed displacement in Z (mm) amounts to a maximum of 16%. The difference between vertical and line of sight (LOS) has not been significant in most cases, mainly when deformation values are neglectable. However, it is recommended to take this difference into account for an accurate comparison with ground instrumentation data that measures in the orthogonal plane.

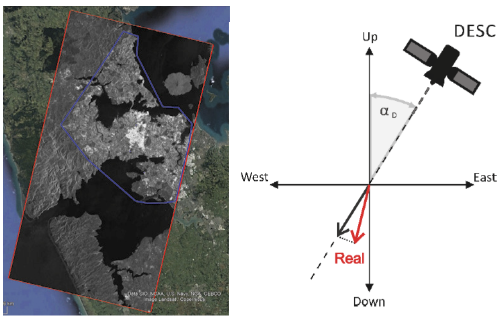


Figure 1. SAR image coverage and properties

When using high-resolution imagery with X-band, the measurement precision of movement in the Line Of Sight (LOS) is between 2 and 3 mm and the precision below 1 meter in the location.

2.2 Climate datasets

From all the available climate observations on local weather stations, soil moisture, temperature and precipitation are proposed as relevant variables influencing soil expansivity (Rogers et al., 1985). Open climate daily observations from the National Institute of Water and Atmospheric Research (NIWA) weather stations were downloaded for three climate variables (temperature, precipitation, and soil moisture). In Figure 2, data from six urban weather stations is averaged on the SAR imagery acquisition dates to understand the underlying conditions of the InSAR data processing. For precipitation, amounts are accumulated between each pair of SAR acquisitions.

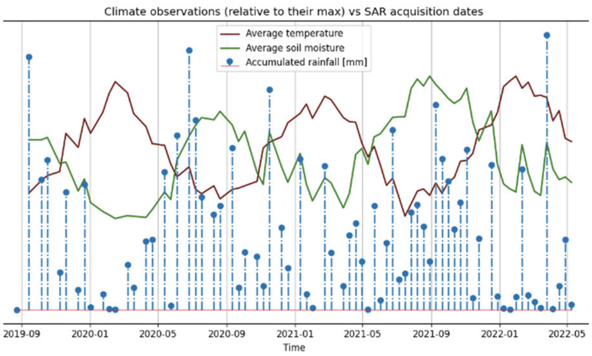


Figure 2. Climate observations vs SAR acquisitions

Soil moisture is influenced by temperature and precipitation seasonal variations. Soil moisture is at its lowest in the *dry* season, where precipitation is low, and the temperature reaches its maximum values, whereas the opposite effect can be observed in *humid* conditions.

2.3 Soil data layers

Available soil cartographies from Landcare Research NZ over the city of Auckland do not provide specific information, as they usually define the urban area as urban soil. Nevertheless, geological maps can give an approximate idea of the type of soil that can be found on the surface.

The geological map of the city of Auckland provided by Manaaki Whenua Landcare Research shows three main formations and all of which may present expansive clay minerals to a greater or lesser extent, as shown in Figure 3.

In the Auckland downtown, the Auckland Volcanic Field formation predominates, and it is characterized by Basaltic and ash deposits that, in seasonally humid climates, typically produce expansive clay minerals.

The East Coast Bays formation also occurs in the downtown and in the north and western regions of the city. This formation is characterized by turbidites that also contain expansive clay minerals.

Finally, the Puketoka formation mainly occurs in the west and south-eastern regions of the city and usually presents high variability in the sediments with occasional clays that may produce expansive patterns. Overall, the geology in Auckland shows that expansive clay minerals may be found all over the city.

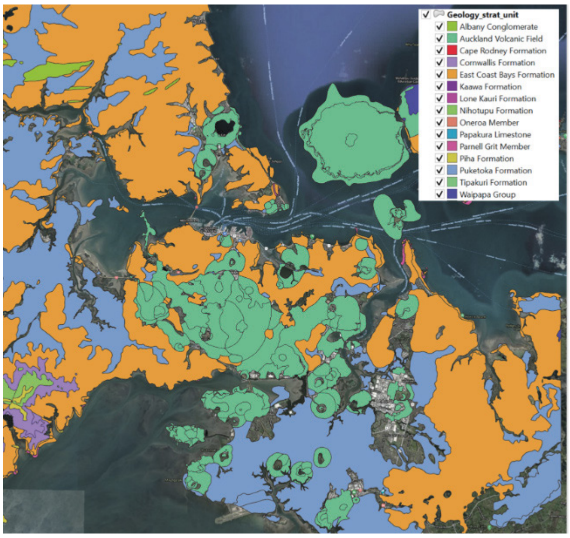


Figure 3. Formations in the Auckland Urban area

2.4 Building footprints

Over half a million building outlines have been extracted from the Land Information New Zealand (LINZ) open dataset archive. The Auckland building footprints (Figure 4) have been obtained from high resolution aerial optical imagery using the latest machine learning roof detection techniques.

The downside of this very precise and updated building footprint polygons dataset is the lack of metadata (e.g. foundation type, year of construction).

Figure 4. LINZ building footprints

2.5 In situ ground instrumentation

The City Rail Link (CRL) consists of twin 3.4 km long tunnels up to 42 metres below the city streets to create an underground rail line linking Britomart and the city centre with the existing western line near Mt Eden.

Sixense deployed and collected data from a deformation monitoring network, including Automatic Total Stations. This network has been operational since May 2020 in the area, and its mission is to provide real-time monitoring data over a valuable group of assets near the construction site. In Figure 5, ATS measurements are compared with Atlas InSAR to show the potential of using both monitoring solutions and provide cross-validation.

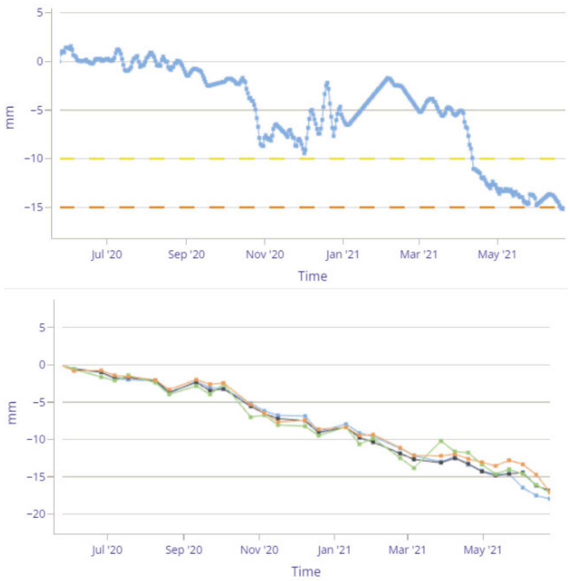


Figure 5. ATS versus 4 Atlas InSAR points

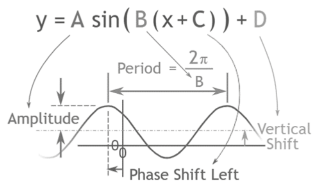
3 METHODOLOGY

The main input for the method developed to map soil expansivity is the ground motion time series obtained with Sixense's Atlas InSAR processing chain. From those, a simple data processing flow has been derived to extract those time series where the soil expansivity pattern is highlighted. This procedure consists of three steps:

First, we need to filter out those time series that do not add any value to the analysis. Noise removal has been done by means of autocorrelation and detecting sudden jumps in the time series related to phase unwrapping errors. A low amplitude filter of ±2 mm has been applied to avoid incurring issues related to the precision of InSAR measurements described in Section 2.1.

After this step, it is necessary to identify those time series with a seasonal component. This is done with sinusoidal curve fitting to the time series data and focusing on yearly periods. Thanks to this fitting, we can clearly distinguish the thermal pattern (T) from dilatation/contraction on metal materials from the seasonal pattern coming from expansive clays (E). By means of linear regression, we can remove the linear trend to focus and quantify only on the seasonal effects. This is especially useful for assessing the intensity of these deformation patterns.

In Figure 6, hotter (and dryer) months are indicated with orange vertical lines and colder (and more humid) months with dark blue:



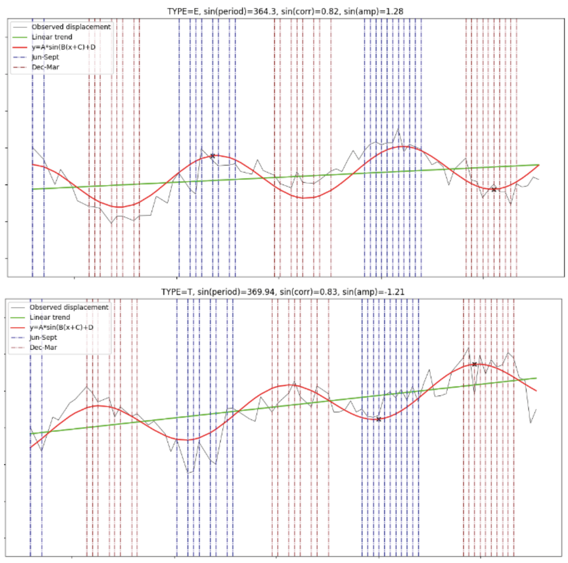


Figure 6. Sinusoidal fitting to ground motion data

Finally, aggregation of those patterns closely related in terms of correlation and distance has been performed to obtain Active Deformation Areas (ADA). This spatial aggregation technique is used to identify those ground deformation time series that represent an active deformation and meet a minimum distance requirement (spatial correlation). It is a technique used in risk assessment from InSAR datasets as described in the article (Navarro et al., 2020). Each ADA consists of a set of ground deformation time series following the same trend together with an envelope polygon identifying the area of influence.

4 OUTPUT DATASETS

After applying the seasonal filtering methods described in Section 3, 379k time series out of the 7.6M are identified as directly related to soil expansivity. From those spatially and temporally related, 61k active deformation areas are delimited. Three different GIS layers have been produced to understand the density, intensity, and possible affectations on buildings of these deformation patterns.

4.1 Density maps

Hotspot raster maps are produced with the location of the seasonal motion point coordinates related to soil expansivity. Due to the surface reflectivity properties, this map will have no data regions on vegetated and water surfaces. As stated in Section 2.3, expansive clays are present in all Auckland geological formations, and therefore patterns are spread across the city. In Figure 7, we can distinguish different concentration of expansive soil deformation patterns.

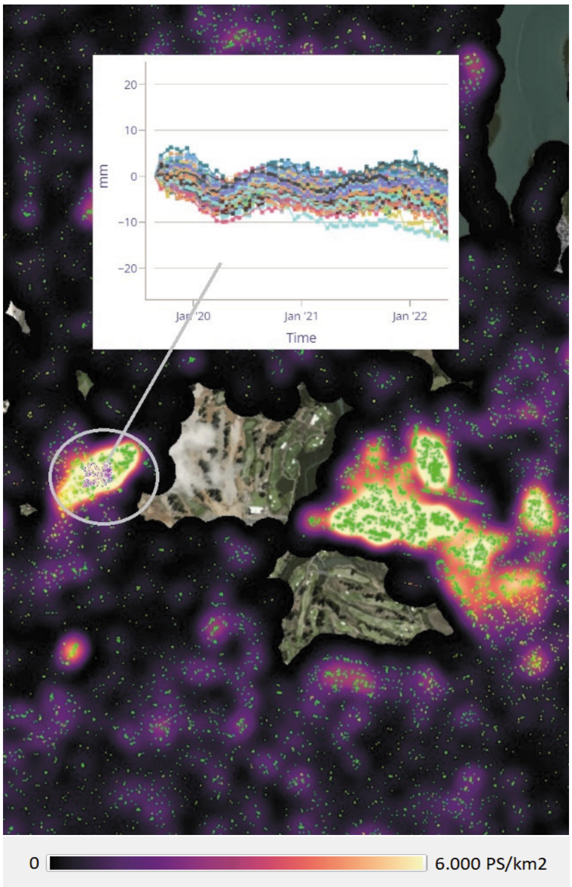


Figure 7. Expansive soil pattern density map

4.2 Intensity maps

Considering the amplitude of the sinusoidal fit without the linear trend of the time series, we can produce maps showing the variations in the intensity of these patterns. This map is useful in general/relative terms but must be considered an approximation since:

- We have considered satellite line-of-sight (LOS) ground motion observations as described in Figure 1 with the precisions described in Section 2.1.
- Amplitude of the sinusoidal fit is generally lower than the real/observed time series. The fitting acts as a smoothing function that reduces noisy values and helps in the interpretation of the results.
- In Figure 8, we can observe a histogram of the different sinusoidal amplitudes. Warmer colours are used to represent higher amplitudes.

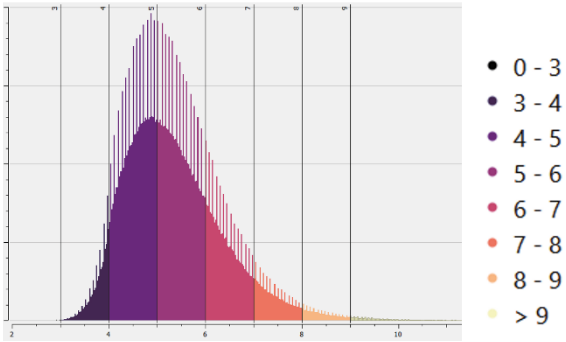


Figure 8. Histogram distribution of amp(sin(x)) approximation in mm

In the next Figure 9, we see a sample of the intensity map highlighting a set of time series exhibiting strong expansive soil deformation patterns.

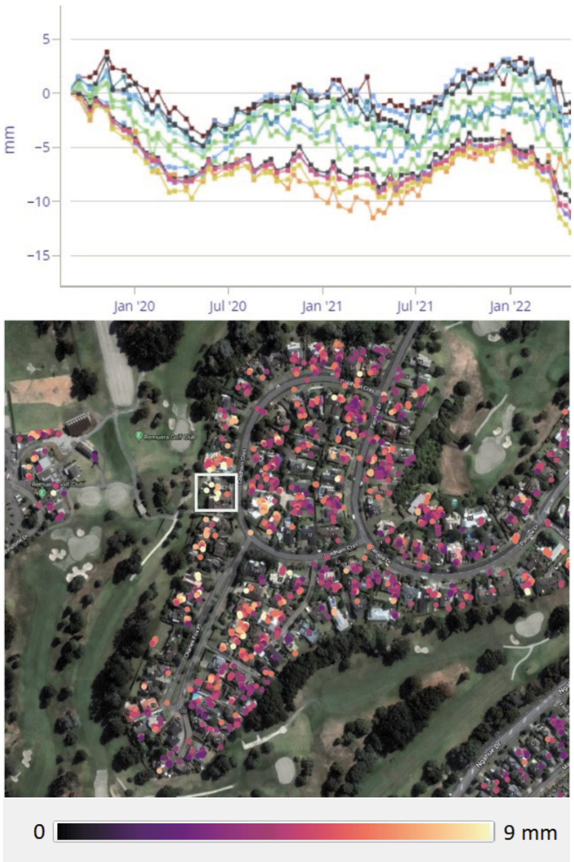


Figure 9. Expansive soil pattern intensity map

4.3 Affected buildings

To highlight those buildings potentially affected by soil expansivity, a vector layer has been produced with a subset of the half million building footprints described in Section 2.4.

Active deformation areas intersecting with the footprints and isolated points falling inside the building have been considered, leaving 133k potential buildings affected (of the total 450k buildings). From those and focusing on the mean yearly amplitude histogram, we can conclude that most of the movement amplitudes are lower than

8 mm, leaving less than 2000 buildings affected by amplitudes higher than that threshold.

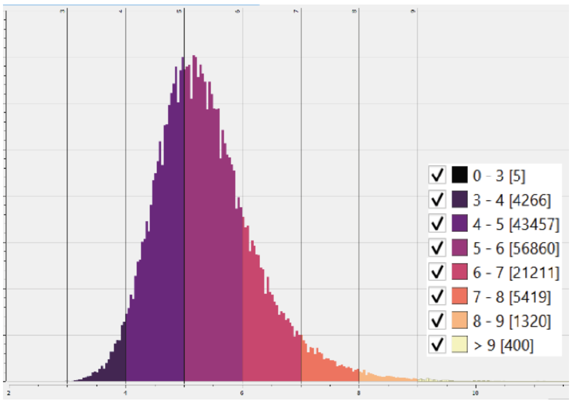


Figure 10. Histogram of amplitudes on buildings

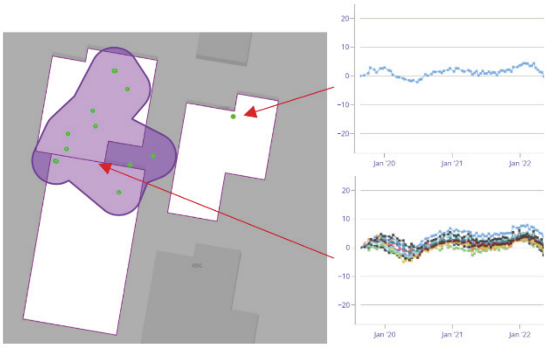


Figure 11. Buildings selection by points and ADAs

5 CONCLUSION

Expansive soil mapping on a large urban area such as the city of Auckland (450 km^2 and 450k buildings) would demand a considerable effort for in situ techniques.

This paper presents a remote sensing approach based on InSAR time series that can be used as a tool to obtain a first scan and mapping of seasonal patterns that enables control and focus on the assets potentially at risk. Due to its backscattering properties, man-made structures such as buildings typically produce high point densities, enabling this kind of analysis for intensity maps. On the other hand, vegetated areas such as parks remain unmapped due to coherence loss, and this poses a challenge in creating density maps.

A seasonal data filter has been designed and adapted to obtain those time series that show the seasonal pattern. Taking the coordinates and movement amplitude, user-friendly output map layers have been produced (density, intensity, buildings) and displayed in the Beyond Monitoring platform.

Information on urban soil is very difficult to obtain, and the presented approach will help the Auckland Council determine the intensities of expansive soil patterns across the city in preparation for the climate challenges ahead.

6 ACKNOWLEDGEMENTS

The presented article has been done in close collaboration with the Auckland Council. Climate change adaptation is a reality, and many urban areas worldwide are facing the same issue as the one of this presented use case.

This work has been supported by the AI Program from Leonard, the innovation platform of VINCI. We would like to thank the coaches and mentors (especially Louis Dumont) for allowing us to develop, benchmark and improve the algorithms used during this case study.

Finally, we would like to thank our colleagues of Sixense Oceania for their work enabling this interesting project to come through and supplying the ground instrumentation data.

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SESSION 2

DATA MANAGEMENT AND DIGITAL TWINS

Photo courtesy of Narges Khajavi (ATC Williams)

Breaking new ground for Geotechnics in Infrastructure Delivery

E. Bugeja[1], B.Eng (Mech), Chairman BuildingSMART Australasia

[1]BuildingSMART Australasia; email: ericbugeja@buildingsmart.org.au

ABSTRACT

Technology has had a significant impact on the information available to us in all aspects of our lives. This has allowed us to do things that were previously not possible. Both in our personal lives and in our professional lives.

In the Engineering and Construction sector, technology has allowed for new ways to design and collaborate. Working in a BIM/Digital Engineering based environment changes the way information can be shared and connected. This way of working is mature in the vertical infrastructure (buildings) space and is now also becoming standard practice on civil linear infrastructure projects.

This paper will explore how the technology has changed and what this means for the Geotechnics profession. It will outline how geotechnical engineers take advantage of the information that is being produced. Conversely, how they can utilise similar technology in the way they analyse and communicate their own technical information. It will highlight how technology will allow for improved collaboration between Design, Construction and Geotechnics.

Keywords: Digital Engineering, BIM, interoperability

1 INTRODUCTION

Technology has enabled the mass digitisation of data. The benefits of producing, storing, analysing, and delivering information in a digital form has widespread opportunities for any business operation. With the appropriate structure and control of this data, project teams will realise:

- **Greater collaboration**, since all team members interact through a common data environment, this consequently becomes the single point of truth
- **Better decisions**, as all stakeholders have access to the most current available information at any point in time
- **Greater efficiency**, as information can be reused throughout the project lifecycle and recycled for subsequent projects
- **Improved value** for our clients, deliverables produced have greater intelligence and can be used beyond design and construction.
- **Improved Quality**, structured and controlled data allows greater traceability. Quality control can take place dynamically rather than at discrete checkpoints

To realise these benefits, project stakeholders must ensure that we standardise and clearly articulate the way we receive, manage, and produce information. This means ensuring that we follow standards on how we request information from our supply chain and clients. Information produced by any project participant should allow for the information to be used beyond a single purpose and enable interoperability with multiple systems. Teams must be structured in a way that promotes open data exchange and collaboration, therefore enabling more collaborative workflows.

This paper will explore how this way of working is impacts the geotechnical discipline and the opportunity it provides for greater collaboration and improved outcomes on infrastructure projects.

2 BACKGROUND

2.1 Industry Change

The way that Engineers and Architects design infrastructure has changed over the years due to the availability of design software that can develop designs parametrically based on engineering rules. Geometry can be rapidly generated in 3D space with rich data linked to the 3D objects. Multiple disciplines can regularly federate these models to resolve issues such as clashes or more rapidly optimise the design. The digital information developed through this design process can be passed on and built upon through various stages of the project lifecycle, avoiding rework between phases. The above process is normally what is referred to as Building Information Modelling (BIM). The use of BIM started in the vertical infrastructure space (Buildings) and its use is more prevalent in this space. As applications have matured in the civil space, linear infrastructure is starting to reach similar levels of maturity.

2.2 Digital Engineering

The term Digital Engineering extends the process of BIM to the integration of other engineering-based data sources throughout the project lifecycle. This could be through systems such as Geospatial Information Systems (GIS) or other design and construction-based information and systems.

2.3 Traditional Information Exchange

As the application of Digital Engineering is becoming more prevalent in the civil infrastructure space, the opportunities to improve the design and construction process are vast. Engineers who design infrastructure rely heavily on the data they have available to them at any point in time. Geotechnical information is critical to many designs, particularly in the design of subterranean structures such as tunnels.

Traditionally, engineers have relied on geotechnical reports being provided to them in analogue forms such as reports and borehole logs. Conversely, the geotechnical teams rely on instruction from the engineering team to determine areas of study and required investigation. The collaboration between the Geotechnical Engineer and Design Engineer tends to be transactional, based on the analogue data being provided to the design engineer to be manually checked against the design.

3 OPPORTUNITY TO IMPROVE COLLABORATION

3.1 Advances in digital geotechnical interpretation

In the same way that Engineering design software has advanced, so has the software used for undertaking geotechnical analysis. Technologies are now available that can assist the Geotechnical Engineer to produce 3D interpretations of investigative data.

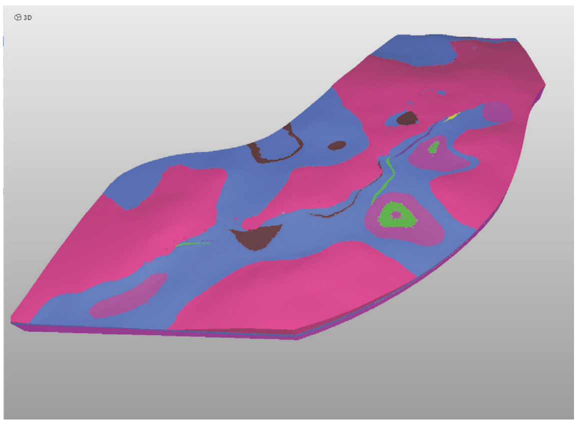

Figure 1. 3D Geotechnical model exported from Leapfrog

Secondly, technologies are available that allow analysis of the impact of a structural design based on ground conditions utilising the available 3D design geometry.

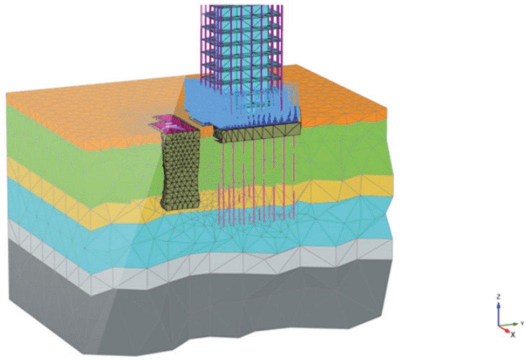

Figure 2. Analysis model in Plaxus (Courtesy of Bentley Systems)

3.2 Current Challenges

Despite many Geotechnical engineers taking advantage of 3D based analysis and interpretive tools, they still tend to sit in isolation from the broader design team. Design models that are received by a Geotechnical team, tend to require manual conditioning to be imported and utilised by 3D analysis software.

3D analysis or interpretive outputs are typically reviewed in isolation due to challenges in bringing the results into the design authoring tools that allow development of the design within the geotechnical context.

As this has been the case, historically the geotechnical teams are not typically setup as part of ISO 19650 (ISO, 2018) based workflows that allow the design team to federate and regularly collaborate on 3D design data through a Common Data Environment (CDE). These environments are setup with areas known as Work in Progress (WIP), Shared and Published with data moved between environments based on Suitability codes.

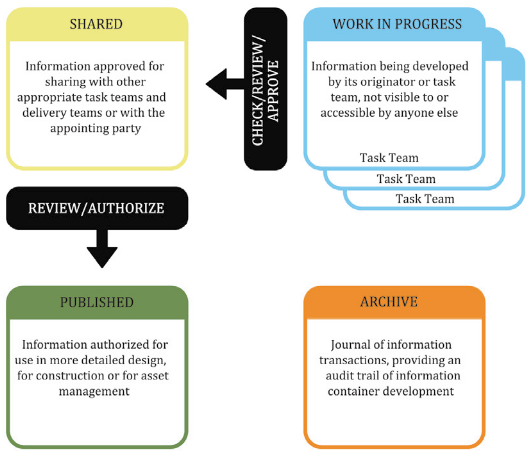

Figure 3. ISO19650 Common Data Environment workflow

3.3 Advances in interoperability

Interoperability is the ability to exchange data between software applications, which eliminates the need to manually copy data already created by one application. In the construction industry, exchanging data is of paramount importance due to its high level of fragmentation. Data can be exchanged using Application Programming Interfaces, proprietary formats of software providers, or vendor-neutral formats (Eastman et al., 2011).

The latter ones play the integral role in the construction industry because they preserve fair competition on the software market and to prevent the market from vendor lock-in (Borrmann et al., 2018).

Developed and maintained by buildingSMART International, IFC is the dominant international standard for exchanging BIM data in the built environment.

buildingSMART liaises with the International Standardization Organization ISO, regional and national standardization bodies for developing international, regional, and national standards.

The IFC data model has traditionally focussed on building projects, but the latest version (IFC4.3) has been enhanced to better support Infrastructure projects, in particular the domains of Roads, Rail and Ports and Waterways.

As part of this focus, the IFC4.3 schema has been developed as an open format that will allow for the exchange of geotechnical information, both data and geometry is then able to be exchanged between software vendor solutions used in various engineering disciplines. This will allow for improved collaboration between geotechnics and other disciplines.

4 OPPORTUNITIES IN GEOTECHNICS

The availability of an open format that will allow for an improved exchange of information between geotechnics team and the design team will enable improved workflows and collaboration by: -

- Allowing the geotechnics team to import the latest 3D design into their own software for the purposes of geotechnical interpretation and analysis.
- Allowing the design team to import updated geotechnical data in a 3D format directly into their BIM based design software, allowing the designer to optimise the design within the context of the geotechnical interpretation or analysis.

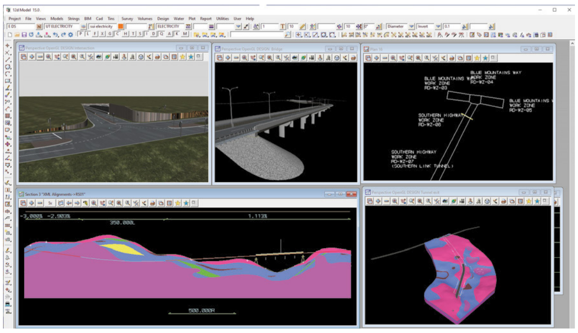

Figure 4. Example of Geotechnical site interpretation shown within civil design software using ifc exchange

Ultimately, this improved workflow will allow for a future of computer based iterative optimisation of a design directly in their design software based on various constraints which would include geotechnical information.

An example could be a road alignment that could be generatively developed based on constraints such as cut and fill balance, geotechnical conditions, environmental conditions, drainage/flood modelling, material usage, etc.

4.1 Defining requirements

To successfully utilise a more collaborative workflow between geotechnical and design, it will be important that the geotechnical team provides input into information requirements defined at the beginning of project. This will ensure that the design data being provided by the design teams are structured and exported in a way that ensures that it can be easily consumed by the software used for geotechnical interpretation and analysis. Conversely, the design team should also document their requirements for the data they require to be exported on a regular basis from the geotechnical teams for use within the design context. These requirements are usually documented in what is referred to in ISO 19650 (ISO, 2018) as an Information Delivery Plan, the plan defines the level of detail (LOD) and Level of Information (LOI) required by each stakeholder at various project stages.

4.2 Collaborative working

Aside from defining requirements, it is also important that geotechnical teams are setup on project within the Common Data Environments to enable collaborative workflows with the broader team. This will ensure that they will be getting regular design updates (typically weekly) as well as ensuring that the design teams are also getting the latest geotechnical information within the 'Shared' environments.

5 CONCLUSION

The advancement of technology and the introduction of Open formats such as IFC now mean that improved digitally based information exchange can occur between geotechnical, design and construction teams. This information exchange includes both 3D geometry, and the metadata linked to the geometry. This will allow for geotechnical teams to become more embedded in the development of designs that rely on geotechnical data, ensuring improved collaboration between project participants.

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openBIM standards for geotechnic information exchange

J. Mirtschin[1], B.Eng (Hons) / B.Sci

[1]Geometry Gym Pty Ltd, 91 Model Lane, Port Fairy, Victoria, Australia; PH (+61) 5568-1163; email: jonm@geometrygym.com

ABSTRACT

The Industry Foundation Classes (IFC) is an ISO standard published for information exchange in the built environment. The standard has recently been enhanced to better support geotechnic objects and relationships, particularly in the context of infrastructure projects. This paper will explain the purpose of openBIM, the collaboration that it can facilitate, and the benefits derived from openBIM exchange. The paper also offers a description of IFC improvements to facilitate geotechnic collaboration, such as recording site investigation and interpretation. Relevant existing concepts in IFC include classification, geometric description, spatial containment, property sets and quantity take offs.

Keywords: openBIM, IFC, BIM, Interoperability, Standards, Software

1 INTRODUCTION

Project actors (designers, surveyors, builders, and technologist) are increasingly authoring digital models of the built environment. Commonly referred to as Building Information Models (BIM), they are typically stored in specialist, bespoke databases or data models unique to the preferred software of the actor.

Digital collaboration is constrained when not facilitated by openBIM data exchange, as the recipient would require the same specialist software to access the model information.

This paper outlines the latest improvements to the openBIM data model “Industry Foundation Classes” (IFC).

Created and maintained by buildingSMART International, the Industry Foundation Classes are the pre-eminent standard for exchanging information for the construction industry.

The Industry Foundation Classes IFC specification is registered as ISO 16739.

To date, the Industry Foundation Classes data model has specifically targeted building projects, but the latest version (IFC4.3) has been enhanced to better support infrastructure projects, in particular the domains of Roads, Rail and Ports/Waterways.

This paper will identify enhancements of the standard that relate to the field of geotechnic information exchange.

2 BACKGROUND

2.1 Interoperability

Interoperability is the ability to exchange data between applications or process and make use of the information. Without interoperability, users must manually transcribe information from one application to utilize in another application. In the construction industry, exchanging data has always had high importance due to the complexity of projects with specialist actors working in tandem to achieve complex outcomes. Data can be exchanged using Application Programming Interfaces (API), proprietary data formats of software vendors, or neutral formats or standards (Eastman et al., 2011).

Open data standards have an important role in the construction industry because they facilitate innovation and competition within the software market, and protect the market from vendor lock-in (Borrmann et al., 2018). Open data standards reduce risk of loss of project data over the long term when a vendor might be lost to industry for any number of reasons.

A compelling explanation on the importance of interoperability is found a promotional video (Bourke J., 1994)

2.2 Compatibility

The Industry Foundation Classes support multiple domains of the built environment, to support information exchange across these domains. Domain focused data models are typically more detailed and efficient at exchanging more specialist data within the domain.

Alternative standards such as AGS Format (including interpreted ground models) will likely remain a superior choice for information exchanges within geotechnical participants, and IFC more typically used for exchanges outside the geotechnical domain.

Reuse of common concepts across multiple domains can constrain most efficient modelling practise specific to an individual domain.

It is expected that industry bodies, including AGS, OGC and buildingSMART will endeavour to ensure high compatibility between these data model alternatives.

3 INDUSTRY FOUNDATION CLASSES

3.1 Object Oriented Classes

IFC is a data model using an object-oriented approach, with single inheritance. Each concept has attributes and relationships. Geometric representation and semantic description in the built environment can be nominated using these attributes and relationships.

3.2 Serialization

There are two official text serializations of IFC data sets for saving content to text, "Standard for the Exchange of Product model data" (STEP) and Extensible Markup Language (XML). Other technologies have been utilized, such as JavaScript Object Notation (JSON) or Structured Query Language (SQL) but are not yet published as official technologies.

STEP physical files is the typical and dominant serialization, utilizing a ".ifc" file extension. It is an efficient text serialization using line number references for relationships to other concepts (Figure 1).

XML is a more modern serialization which is more easily understood by humans due to attribute name identification and nesting (Figure 2). The file size suffers from the additional attribute name identification and the use has been constrained due to the file resultant file size.

Text file serialization offers long term data security as it is least likely to become a lost technology. Any human user can open a STEP or XML file in notepad or a text editor, however, a human may have difficulties in interpreting it correctly. Text serialization is less efficient than binary encoding. Resultant file sizes can be unacceptable, but the ifc file can be compressed into a zip file and saved with a file extension "ifczip". Zipping an IFC file can achieve file size reductions in the order of 20% to 50%.

```
#22=IFCCARTESIANPOINT((0.,0.,0.));
#23=IFCAXIS2PLACEMENT3D(#22,#21,#20);
#24=IFCLOCALPLACEMENT($,#23);
#25=IFCSITE('02FkWai_532fIiNwUn3e4o',#9,'Site_1',$,$,#24,$,$,.ELEMENT.,$,$,$,$,#2);
#26=IFCPROJECT('2Z5iVSY21A5BDQap9aMYN_',#9,'Alignment-Aplitop-1','',$,'',$,$,#14);
#27=IFCRELAGGREGATES('1lAfS8jBjDmR$WPSQBvIFd',#9,$,$,#26,(#25));
#28=IFCROAD('1YqzGB3UT6iv9rSn3ZiChz',$,'Some road',$,'IfcRoad',$,$,$,.COMPLEX.,$);
#29=IFCRELAGGREGATES('3ovWpuZRj6SgHHC$37HFXe',#9,$,$,#25,(#28));
#30=IFCALIGNMENT('0jf_kMZ$nCZvfW8BjN2_gf',$,'Alignment1',$,$,#24,$,$);
#31=IFCALIGNMENTHORIZONTAL('0IfcjsZ$nE2v5xfcXk_T0i',$,'Alignment1',$,$,#24,$,0.);
#32=IFCRELNESTS('0EnDoLEyj8c9WsNDJc2NsV',#9,$,$,#30,(#31,#80));
#33=IFCALIGNMENTSEGMENT('0wbpE5Ppv2h8YpAmz1jvCt',$,$,$,$,#24,$,#35);
#34=IFCCARTESIANPOINT((335085.95782221,4084594.13214501));
#35=IFCALIGNMENTHORIZONTALSEGMENT($,$,#34,6.24482463878552,0.,0.,10.,$,.LINE.);
#36=IFCALIGNMENTSEGMENT('3GuFB_j892ogFRXkUKFjOT',$,$,$,$,#24,$,#38);
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#38=IFCALIGNMENTHORIZONTALSEGMENT($,$,#37,6.24482463878552,25.,25.,39.84063746,$,.CIRCULARARC.);
#39=IFCALIGNMENTSEGMENT('3d1079w0TExBB5SBeqnxzs',$,$,$,$,#24,$,#41);
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```

Figure 1. Sample of STEP Physical File serialization

```
<IfcRelContainedInSpatialStructure GlobalId="3_GokZQqz3AfM$ckWYLAn4">
  <RelatedElements>
    <IfcAlignment GlobalId="0jf_kMZ$nCZvfW8BjN2_gf" Name="Alignment1">
      <IsNestedBy>
        <IfcRelNests GlobalId="0EnDoLEyj8c9WsNDJc2NsV">
          <OwnerHistory xsi:nil="true" href="i9" />
          <RelatedObjects>
            <IfcAlignmentHorizontal GlobalId="0IfcjsZ$nE2v5xfcXk_T0i" Name="Alignment1"
            StartDistAlong="0">
              <IsNestedBy>
                <IfcRelNests GlobalId="1ybgUMDlb60x4Z8Z8W39S$">
                  <OwnerHistory xsi:nil="true" href="i9" />
                  <RelatedObjects>
                    <IfcAlignmentSegment GlobalId="0wbpE5Ppv2h8YpAmz1jvCt">
                      <ObjectPlacement xsi:nil="true" href="i24" />
                      <GeometricParameters xsi:type="IfcAlignmentHorizontalSegment" StartDirection="
                      6.24482463878552" StartRadiusOfCurvature="0" EndRadiusOfCurvature="0"
                      SegmentLength="10" PredefinedType="line">
                        <StartPoint xsi:type="IfcCartesianPoint" Coordinates="335085.95782
                        4084594.13215" />
                      </GeometricParameters>
                    </IfcAlignmentSegment>
                    <IfcAlignmentSegment GlobalId="3GuFB_j892ogFRXkUKFjOT">
                      <ObjectPlacement xsi:nil="true" href="i24" />
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                      6.24482463878552" StartRadiusOfCurvature="25" EndRadiusOfCurvature="25"
                      SegmentLength="39.84063746" PredefinedType="circulararc">
```

Figure 2. Sample of Extended Markup Language Serialization (XML)

3.3 Model View Definitions

IFC is a comprehensive data model that requires considerable effort to support in full. It covers several domains and disciplines, and, for example, an application or process considering cost or quantity take offs for a model, does not require to interpret structural analysis features of a model (i.e. restraints or load features).

To permit specific and focussed implementations, a model view definition (MVD) relates to a use case of BIM interoperability or exchange. This will nominate concepts that are required to facilitate this exchange, and concepts that are prevented from facilitating the exchange. This includes constraint on geometric descriptions such as non-uniform rational b-splines being prevented or only triangulated face set being necessary.

There are three key model view definitions offered by buildingSMART, with other derivations or alternatives in progress or offered by others. IFC2x3 Coordination view is the dominant MVD still in use today. IFC4 Reference View is a more constrained MVD focussing on visualization or model reference using constrained tessellation geometry definitions. IFC4 Design Transfer View offers semantic or intelligent descriptions of models but has not progressed from a draft status, primarily due to difficulties in agreement on semantic concepts that should or should not be included.

4 INDUSTRY FOUNDATION CLASSES CONCEPTS

IFC has been the standout openBIM data model for buildings in the past decade for buildings. In infrastructure projects, alternatives such as landXML or GENIO. IFC offers several concepts that are not adequately offered in these alternatives and IFC4.3 release improved IFC in many key aspects needed for infrastructure project interoperability.

Detailed explanations of useful BIM concepts contained in IFC are described by Borrmann et al. (2018) and will only be highlighted in this paper with infrastructure context for insights and comparisons.

4.1 Spatial Structure

Spatial structure within IFC is a project hierarchy about how to locate an element or system. In a building sense, it might be a room (IfcSpace) on a building storey (IfcBuildingStorey) within a building (IfcBuilding) on a site (IfcSite). In an infrastructure sense, it might be on a median (IfcRoadPart) on a corridor (IfcRoadPart), or on a road facility (IfcRoad).

4.2 Material

IFC offers comprehensive standard descriptions and attributes of materials that can be assigned to products (or components of products).

4.3 Geometric Description

A key component to BIM in contrast to alternative Information Models is the expectation that geometric descriptions of products will be provided in typically 3-dimensional context. Actors interacting with BIM can have the ability to traverse the model in a virtual environment.

IFC offers a variety of shape descriptions, including constructive solid geometry, sweeps and NURBS boundary representations. There is a trade-off in that the more complex semantic descriptions seem to have less consistent implementation/support by software vendors.

Most BIM models are dominated by extrusion or primarily faceted geometry, which are the most reliable geometry, or the use case permits approximate faceted geometry to be utilized.

4.4 Placement

IFC elements have a placement/position relative to the element above it in the spatial or aggregation hierarchy.

4.5 Property Sets

IFC objects have attributes that have been identified as standard/typical. Sets of properties are utilized to define element features not nominated as an attribute.

Each IFC specification nominates “standard” sets of properties for common elements (example, fire rating).

Clients / Projects may define or refine the definitions of the properties and at what stages of a project they should be provided.

IFC offers a comprehensive set of measure types to define property values.

4.6 Quantity Sets

An element can be exchanged with a set of derived measures for quantity take off to permit planning or costing of the project. Measures can be of a count, weight, length, area, volume or time nature.

4.7 Classification

IFC offers a built-in classification system to identify concepts (i.e. IfcSite, or IfcStair). This classification is comprehensive, but it is observed that many clients/agencies either have an in-house classification system, or a common classification system such as Uniclass2015.

IFC permits multiple classifications to be nominated within the data model.

4.8 Types and Instancing

IFC offers a relationship of a common type relating to multiple instances. Elements with common characteristics may have the common features applied to a relating type, which is related to all appropriate instances. This includes a common geometry description, which is transformed into individual product placements.

4.9 Systems

A system is an organized combination of related products, grouped for a common purpose or function. As an example, a group of elements might comprise a building foundation system.

4.10 Georeferenced Model

An IFC data set is typically defined in the context of a project local engineering coordinate system. This coordinate system can be located relative to a projected coordinate system such as EPSG.

5 INFRASTRUCTURE ENHANCEMENTS

Over the past 5 years, a core team of stakeholders and technical resources, including software implementers, have worked on advancing the Industry Foundation Classes to better support exchange concepts required for infrastructure projects. This paper will now outline the key concepts revised or added to the standard for release IFC4.3 relating to Geotechnical and other enhancements.

Specialist domain characteristics are less likely to be explicitly nominated within IFC. More typically, the concepts respond to requirements of exchanging data across discipline or domain, for example, from geotechnical interpretation of bearing pressures to engineer or contractor for foundation design or review. Geotechnical interpretation might be overlayed with alignment design software to influence choices with respect to earth conditions encountered (Figure 3).

5.1 Geotechnical

The fundamental product introduced for Geotechnical elements in IFC4.3 is IfcGeotechnicalElement. The first subclass of IfcGeotechnicalElement is IfcGeotechnicalAssembly which in turn has subclasses, IfcBoreHole, IfcGeomodel and IfcGeoslice. The second subclass of IfcGeotechnicalElement is IfcGeotechnicalStratum. The stratum class has predefined types of Solid, Water and Void.

Property sets have been introduced, such as GeotechnicalStratumCommon featuring individual properties Stratum Colour, Is Topographic, Piezometric Head, Piezometric Pressure, Status and Texture.

Another solid stratum property set introduced is SolidStratumCapacity which features properties Cohesian Behaviour, Friction Angle, Friction Behaviour, GrainSize, Hydraulic Connectivity, Load Bearing Capacity, N Value Permeability Behaviour, Poisons Ratio, Pwave Velocity, Resistivity, Settlement Behaviour, Swave Velocty.

An additional solid stratum property set introduced is SolidStratumComposition which features properties Air Volume, Boulders Volume, Clay Volume, Cobbles Volume, Containment Volume, Fill Volume, Gravel Volume, Organic Volume, Rock Volume, Sand Volume, Silt Volume, Water Volume and Composite Fractions.

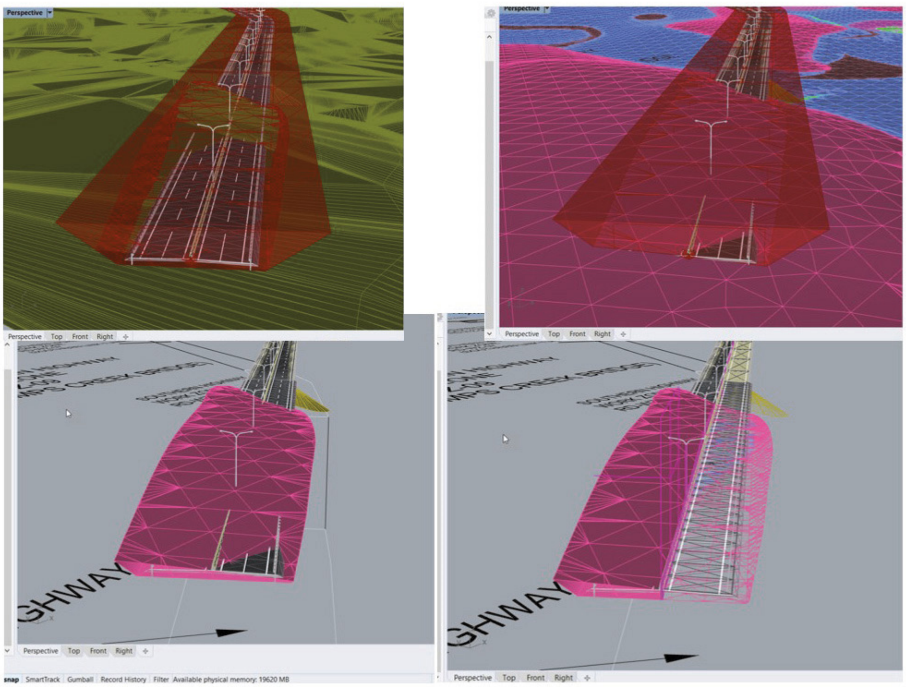

Figure 3. Demonstration of Geotechnical Site interpretation in tandem with linear infrastructure setout.

A property set introduced for boreholes feature properties State, Cap Depth, Cap Material, Filling Depth, Filling Material, Groundwater Depth, Lining Material, and Lining Thickness.

An Uncertainty property set has been introduced to nominate uncertainty in measurements and relate how the uncertainty was assessed.

5.2 Alignments

Alignments are the key concept to define and set out a linear infrastructure project, or also components of an infrastructure project, such as drainage or transmission distribution lines.

Alignments are semantically defined as a series of projections. As a horizontal curve to describe the path. A vertical profile is projected onto the horizontal curve to define elevation and grade. For railway projects, cant (track inclination to compensate for lateral accelerations) can be exchanged.

5.3 Linear Placement

Linear infrastructure projects often utilize a linear placement to locate assets using an alignment with a longitudinal location typically referred to as a chainage or station. Lateral and vertical offsets from this linear point can define a product placement. IFC4.3 permits a geometric placement in addition to a semantic relationship.

5.4 Sectioned Shapes

Tessellated or faceted geometry can quickly accumulate file size in serialization, in particular when writing to text. Sectioned shapes have been introduced with other advanced sweeps to facilitate efficient geometric description of rails or other longitudinal geometry. Figure 4 visualizes the sectional profile shapes and the resultant surface from the resultant sweep.

5.5 Earthworks

Explicit products have been introduced in IFC4.3 to nominate earthworks elements, including cut and fill elements with volumes.

5.6 Classification

Classes have been added to the domains for Rail, Road, Ports/Waterways and common infrastructure projects to denote facilities, facility parts and the elements that comprise them. This includes spatial breakdown as a combination of longitudinal, lateral and/or vertical hierarchy.

6 FUTURE EXTENSION OF IFC

The IFC4.3 extension project did not cover all the desired exchanges for infrastructure and geotechnical requirements. At the time of writing this paper, a project to further enhance IFC4.3 with a focus on tunnels was nearing conclusion, with an anticipated IFC4.4 release to be published. This includes refinement of concepts such as geo referencing model placement.

As IFC4.3 is implemented, supported and utilized, it is expected that other improvements will be identified and submitted to buildingSMART for consideration in future versions of the standard.

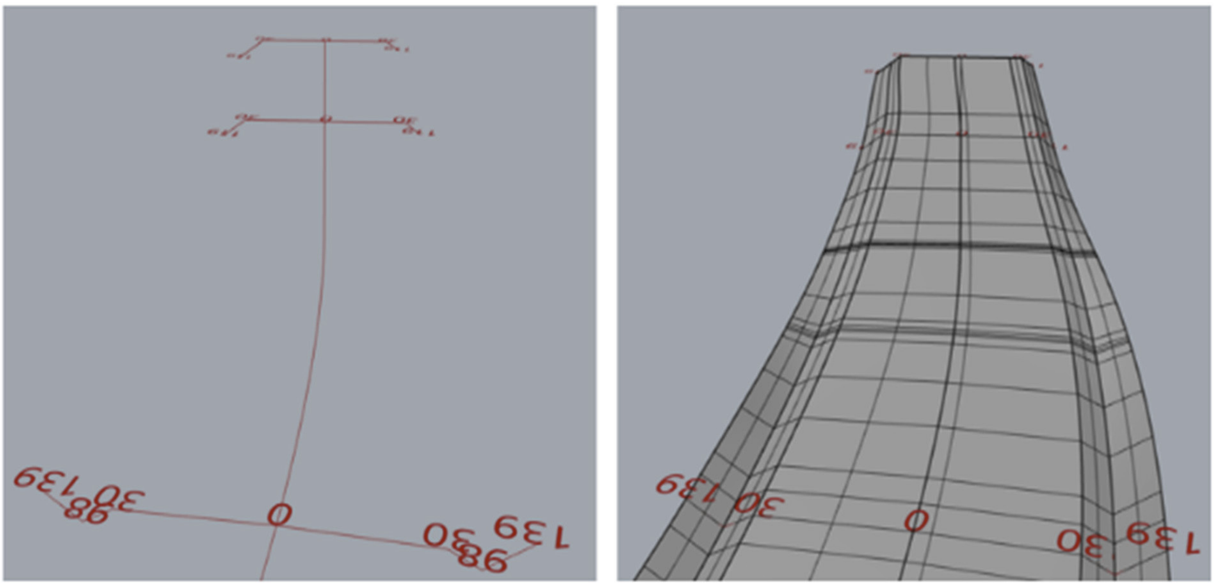

Figure 4. Demonstration of sectioned shape to nominate geometric road surface.

7 CONCLUSION

IFC offers a comprehensive data model to facilitate data interoperability for the built environment, and recent IFC4.3 improvements make it a first class solution for infrastructure projects. Building projects will also benefit from improvements relating to geotechnical data exchange.

IFC might not be the data model of choice for advanced interoperability between specialist geotechnical software or processes, although it can facilitate that with bespoke or agreed properties.

IFC is a first class solution for data interoperability for cross discipline or domain information exchange.

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Implementation of the AGS geotechnical data transmission format: the Brazilian experience

D. G. Yanez[1], A.M. ABMS and F. A. G. V. Reis[2], Pres. FEBRAGEO

[1]PhD Candidate, Institute of Geosciences and Exact Sciences (IGCE), Department of Applied Geology, São Paulo State University (UNESP), 1515, 24A Avenue, Bela Vista, Rio Claro, São Paulo, Brazil, CEP 13506-900; email: diego.gazolli@unesp.br; Experienced Engineer, ATC Williams, 222 Beach Road, Mordialloc, VIC, 3195; PH (03) 8587-0900; email: diegog@atcwilliams.com.au
[2]Associate Professor, Institute of Geosciences and Exact Sciences (IGCE), Department of Applied Geology, São Paulo State University (UNESP), 1515, 24A Avenue, Bela Vista, Rio Claro, São Paulo, Brazil, CEP 13506-900; email: fabio.reis@unesp.br; President of the Brazilian Federation of Geologists (FEBRAGEO).

ABSTRACT

In the technology age, when cloud computing and artificial intelligence are prominent, the Association of Geotechnical and Geoenvironmental Specialists (AGS) digital data format gains its greatest importance. This paper presents a case study using the AGS digital transmission standard for geotechnical investigations in Brazil. The AGS data transmission standard developed in the United Kingdom is discussed with reference to reporting qualities in terms of flexibility and robustness. Next, two sectors in Brazil that started using the AGS format before it was officially implemented in the country are presented. It includes highways operated by private companies and academic researchers focused on the application of Artificial Intelligence and Data Science techniques to a national geotechnical investigation Big Data. Hence, the urgent need to implement a standardized format for geotechnical data transmission; as well as the opportunities that it presents, such as the integration with Building Information Modelling (BIM) software. It includes information for maintenance, operation of infrastructures and the development of automated geotechnical correlations. In conclusion, the case study indicates that, if the digital demands are not met by the official standardization in Brazil of geotechnical data transmission, private and scientific researchers may force this to occur through widespread use.

Keywords: AGS format, geotechnical data, data exchange, regulation

1 INTRODUCTION

In the last few decades, computer technology has profoundly affected all human activities, with significant impacts on communication, transmission, and storage of data. Engineering projects have gradually evolved from manual processes to Computer-Aided Design (CAD). Analysis is routinely performed in highly sophisticated data environments, such as Geographic Information Systems (GIS) or BIM.

It is notable that despite geotechnical information being obtained from costly drilling and laboratory operations, in most cases, the data is poorly documented and curated. Moreover, many projects are becoming more complex, and increasingly large volumes of geotechnical investigation data are produced which require geotechnical practitioners to spend more time and effort to assess and employ this information (Zimmermann et al., 2006). Due to the complexity of these projects, many entities share the acquisition, set up, usage, curation, and presentation of geotechnical information as presented in Table 1.

Despite the existence of standardized formats for transmitting geotechnical data, there are still few countries that have regulations or standards that require their application. Instead of focusing on the difficulties of implementing a geotechnical information transmission standard, this article aims to highlight the opportunities that are being envisioned by those who have already adopted this method of data collection and use. To demonstrate the effectiveness of such a system, a case study is presented on how the Brazilian geotechnical community has started using the format developed by the Association of Geotechnical and Geoenvironmental Specialists (AGS) without official regulations being implemented.

Table 1. Geotechnical entities involved in site investigation and their usage. (Mokarram 2010)

Data usage types	Project Engineer	Field Engineer	Driller	Lab Technician	Data Owner (Client)	Data User (researcher analyst)	Public Agencies	Private Agencies
Generating Data								
Assembling Data								
Utilizing Data for Design/Calculation								
Archiving Data								
Reporting Data								

2 BACKGROUND

2.1 Geotechnical data transfer formats

2.1.1 General

A brief description of the Geotechnical Data Transfer Formats will be given and is not intended to be exhaustive. Two of the most successful efforts that are publicly available for practical application are AGS and DIGGS.

2.1.2 AGS Format

In the early 1990s, geotechnical professionals in the United Kingdom realized the need to standardize digital information from soil surveys and tests and proposed the AGS data transmission format in 1991. It was one of the first noticeable data representations for geotechnical information and is still the most successful information release format accepted by the geotechnical community. Some of the key features are: American Standard Code for Information Interchange (ASCII) text file format, data dictionary, data groups and identifiers, and units. (Mokarram, 2010)

Currently, the AGS format is in version 4.1.1 and is compatible with numerous sophisticated modelling software, including AutoCAD Civil3D Geotechnical Module and some of the Building Information Model (BIM) design software already available for professional usage. (AGS, 2022) Geotechnical interpretation software such as gINT, Rockworks, CPT-Pro and HoleBASE are compatible with AGS.

2.1.3 DIGGS Format

On the other hand, the Data Interchange for Geotechnical and Geoenvironmental Specialists (DIGGS) format began development in 2005 through the joint United States effort of the Federal Highway Administration (FHWA) and the Ohio Department of Transport (DOT) with support from universities and industrial partners. This format has an eXtensible Markup Language (XML) schema and Geography Markup Language (GML) compliant data dictionary. The first DIGGS version 1.0.a was published in 2008 and the most current production version 2.5.a was published in 2020 (Merklin, 2016).

It is, however, understood that DIGGS is a very complex format compared to AGS and its initial versions had many structural inconsistencies and validation problems (Mokarram, 2010).

2.2 Benefits of geotechnical data interchange

The use of a standardized format for transmitting information is important to minimize errors in data re-entry of values, reduce risks, and maximize efficiency in geotechnical assessments. For example, single data entry mitigates errors during information transcription; data entered at the source is available throughout the project lifecycle; there is assurance that the information is consistent and can be used in a variety of applications (Malanconi et al. 2018).

Deaton (2018) summarizes that the application of either geotechnical information formats (either AGS or DIGGS) provides financial benefits to all involved parties: Owner, Consultant and Contractor. Deaton (2018, p. 5) stated that "(…) the typical consultant is re-inputting subsets of the same data between 10 and 15 times per project.". Hence, all parties are likely to reduce risk by eliminating vast amounts of data re-entry and more time can be spent performing engineering design assessments versus simply reinputting the same data.

In our current technological age, having geotechnical information in a data exchange format means organizations can use past data as well as combine all types of geotechnical information from boreholes, laboratory testing and in-situ testing together in a practical query process for more advanced analysis, visualization and even data mining applications (Wang et al., 2020).

A little-discussed advantage of using a standardized information exchange format is the quality gain in the structured organization of data. The most current version of AGS is made up of dozens of groups and headings for different types of field and laboratory tests.

For example, the group table that refers to the General data of a Static Cone Penetration Tests (SCPG) campaign has specific fields for the physical description of the tool, nominal rate of penetration, filter material, level and origin of groundwater with additional environmental fields. Although the CPT test is usually presented with a large volume of information in its reports, it is common for suppliers to not record the geometric dimensions of the cone or filter material in their reports. Rarely do companies record how the groundwater used in the interpretation of the test was obtained. Moreover, the interpreted data in AGS are registered in another group table, called Static Cone Penetration Derived Parameters (SCPP), in which the organization that carried out this evaluation should also be input.

Finally, both AGS and DIGGS formats are dynamic products, with constant development and updates. In particular, for the AGS format, Group tables and Headers disclosed in their Data Dictionary are not limiting. Each user can add new fields that they deem relevant, as long as they are duly documented in the transmitted file. This makes the format versatile and amendable to best practices that have not yet been registered in the current version of the data exchange format as well as new geotechnical tests still to be invented.

3 CASE STUDIES

3.1 Highway concessions

The Brazilian Federal Decree number 10.306, 2nd April 2020 established the use of Building Information Modelling (BIM) in the direct or indirect execution of engineering works and services carried out by agencies and entities of the federal public administration (Brazil, 2020). The implementation of the BIM system was divided into three phases. The first being valid from 1st January 2021, for the structural, hydraulic, heating, ventilation and air conditioning and electrical disciplines, the extraction of schedule of quantities and the generation of graphic drawing documentation. The other phases are applicable from 2024 and 2028 and progressively increase the use of the tools and capabilities of BIM models for engineering projects related to federal public administration.

Unfortunately, the implementation of this decree was poorly planned by the government, which did not analyze what measures would need to be taken to ensure that the implementation of BIM models would be full and efficient. One of the parts not evaluated by the federal regulation was how companies would handle geotechnical information in a BIM environment. Proof of this is that two other federal decrees, decree number 9,377 of May 17, 2018, and decree 9,983 of August 22, 2019, were published to implement BIM models but are now invalid since they were replaced by the most current decree (Brazil, 2018; Brazil, 2019).

Knowing the importance that geotechnical information has to the safety and cost of national roads, the private companies that currently operate several road concessions in Brazil united to evaluate which data transmission standard should be adopted by the sector. This evaluation took place through the formation of the AGS Brazil Group (www.padraoags.com.br) in 2018 (Malanconi et al., 2018).

It is noteworthy that the AGS Brazil Group initiative brought together highway concessionaires, geotechnical investigation companies, geotechnical engineering designers and engineering management companies. More than disseminating and adapting the British AGS standard to the Brazilian culture and custom of geotechnical investigation, the AGS Brazil Group has published 4 open-access guidelines for carrying out geotechnical investigations. These guidelines layout:

1) how to specify a schedule and planning of an investigation campaign, including stoppage criteria to be used during drilling,
2) the procedures that must be followed in the execution of the test, including the measurement of coordinates with precision GPS equipment and the photographic record of the test,
3) ways of describing the investigation results, in particular the geological-geotechnical classification of the samples, and
4) delivery of survey results in AGS digital format to the customer.

The group managed to go beyond the initial objective of disseminating the format of electronic transmission of geotechnical information and also published high-quality guidelines based on standard practices in Brazil.

These practices disclosed by AGS Brazil Group contribute to the reliability of geotechnical information, add value to the maintenance stage of the road project (also called 7D of BIM modelling) and can generate a geotechnical database of geotechnical information for the federal government regulatory agency.

It is noticeable in Figure 1 that the recommended photographic record of all samples for each Standard Penetration Test (SPT) in the AGS Brazil Group guideline adds accountability to a geotechnical investigation that follows it.

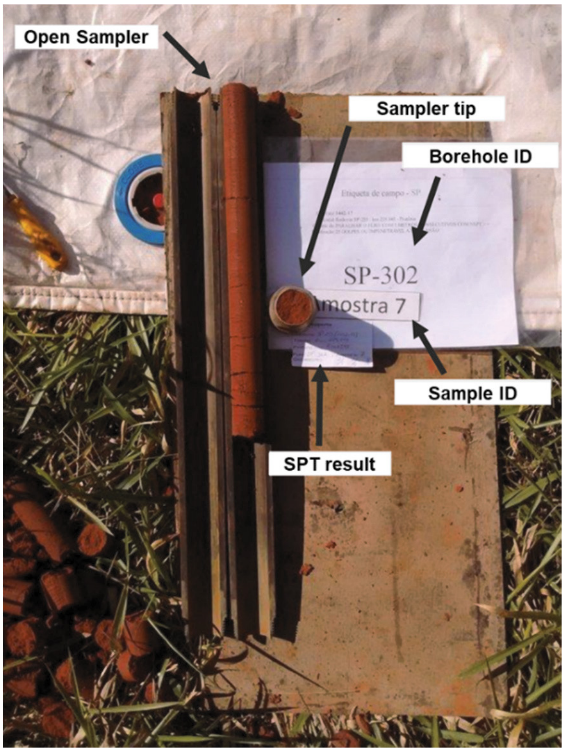

Figure 1. Photographic record of samples during boreholes (AGS Brazil, 2018)

3.2 Scientific Research

In 2018, the geological-geotechnical investigations Big Data startup iSondagens began operating in Brazil aimed at academic research usage. This initiative was inspired by existing geotechnical investigation databases throughout the world, such as DINOloket (Netherlands), GeoIndex (Great Britain), New Zealand Geotechnical Database (NZDB) and InfoSolo (Portugal) (Yanez and Reis, 2019).

By 2021, iSondagens managed to assemble over 1 million geotechnical tests from historical and present boring companies, mostly comprised of SPT boreholes. Figure 2 presents the location of the tests in the Metropolitan Region of São Paulo.

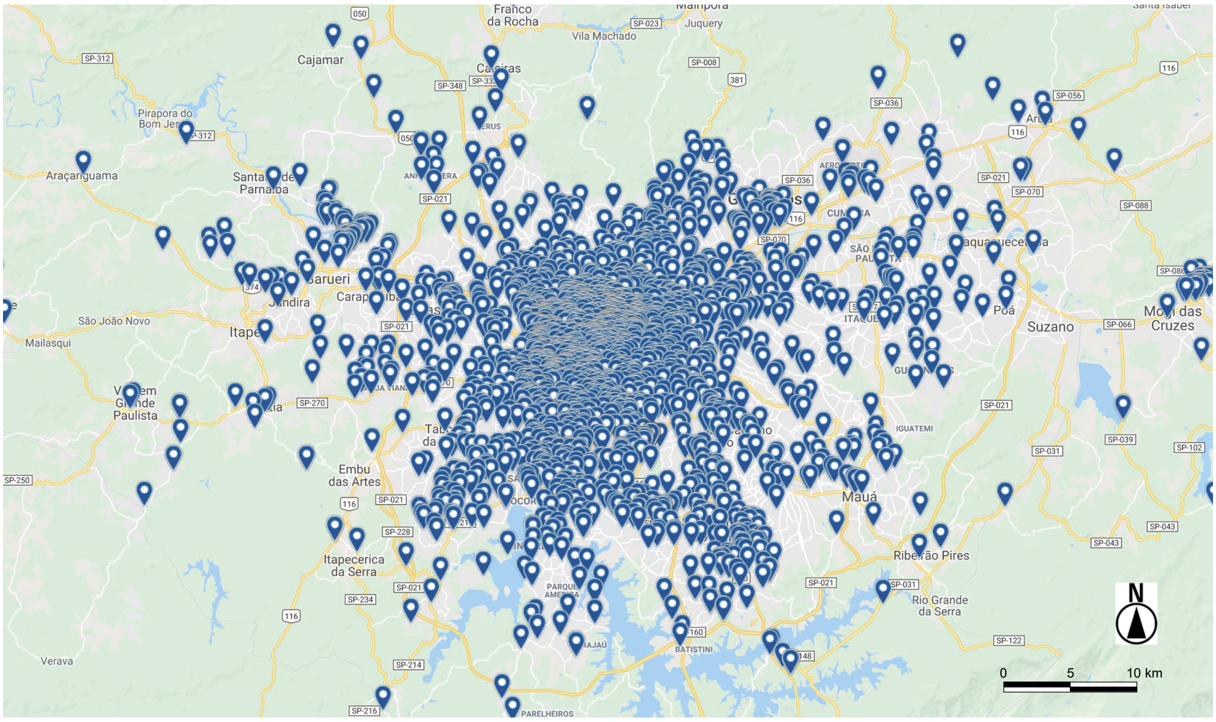

Figure 2. Map of curated by iSondagens historical boreholes at the Metropolitan Region of São Paulo (Yanez and Reis, 2019)

The Brazilian geotechnical investigation industry is particularly interested in Big Data. It is estimated that 6 million meters of boreholes are performed per year for residential construction alone.

One of the challenges faced by iSondagens was the organization of historical boreholes carried out before the publication of the first Brazilian norm for SPT in 1979. According to Yanez and Reis (2019), as the execution of penetration testing in Brazil started in 1939 by the Instituto of Technological Research of the State of São Paulo (IPT-SP), 40 of the current 83 years of geotechnical investigation history in Brazil were carried out without an established standard. This means that during the 1970s there were 3 different samplers for the execution of penetration tests: the sampler developed by IPT-SP itself, the sampler from the Geotécnica company (named Mohr-Geotécnica) and the Raymond sampler, popularized by the famous 2nd edition of the book by Terzaghi and Peck, Soil Mechanics in Engineering Practice in 1968.

The database is composed mainly of historical tests and uses the AGS format expanded to list the aforementioned samplers. For that purpose, information from tests from different samplers could be grouped and interpreted through a simple conversion algorithm based on the existing correlations for these samplers presented in Table 2.

Table 2. Correlations for soil compaction and consistency based on Brazilian historical penetration samplers (Belicanta, 1998 apud Yanez and Reis, 2019)

Soil type	**Relative Density / Consistency**	**IPT Sampler $\emptyset_e$ = 46 mm $\emptyset_i$ = 38 mm**	**Mohr-Geotécnica sampler $\emptyset_e$ = 41 mm $\emptyset_i$ = 25 mm**	**Raymond sampler (SPT) $\emptyset_e$ = 51 mm $\emptyset_i$ = 35 mm**
		Blow count		
Sand and Silty Sand	Very loose	≤ 2	≤ 4	< 5
	Loose	3 – 5	5 – 8	-
	Medium	6 – 11	9 – 18	5 – 10
	Dense	12- 24	19 – 41	11 – 25
	Very dense	> 24	> 41	> 25
Clay and Silty clay	Very soft	< 1	< 2	-
	Soft	1 – 3	2 – 5	< 4
	Stiff	4 – 6	6 – 10	4 – 8
	Very stiff	7 – 11	11 – 19	8 – 15
	Hard	> 11	> 19	> 15

Legend: $\emptyset_e$: External diameter
$\emptyset_i$: Internal diameter

4 DISCUSSION AND CONCLUSIONS

The Brazilian geotechnical practice has been breaking some paradigms in geotechnical investigations. The article aimed to bring some pioneering case studies to the Brazilian scenario.

As presented in the case studies (Malanconi et al., 2018; Yanez and Reis, 2019), the regulation of a data transmission format brings benefits in terms of quality control and quality assurance. These benefits go beyond the mitigation of data re-entry errors, through recommended reliable and verifiable executive procedures and also provide supplementary information that is rarely presented (for example, the origin of the groundwater level and the filter material in CPT).

Having geotechnical information in a data exchange format means organizations can use available past data as well as combine it with recent data for more advanced analysis, visualization and even data mining applications.

Data exchange formats include boreholes, laboratory testing and in-situ testing together to create a practical query process. These tools have the potential to allow geotechnical engineers to evaluate correlations between data variables.

An essential part of developing a geotechnical information database is organizing the data in the same format. National regulation of a geotechnical data exchange format paves the way for the development of geotechnical databases essential for risk management during natural disasters such as landslides and floods.

Finally, due to the growing complexity of engineering projects, the increasingly shorter construction schedules, and the growth in the number of geotechnical investigations; there is great potential for increasing the efficiency of geotechnical engineering projects with computerized tools. This includes the development of dynamic geotechnical models for the design and calculation of earthworks. Together with online instrumentation measurements, it can be envisioned that in the near future, geotechnical engineers would be able to assess earthwork safety and reliability in real-time.

It is then concluded that efforts in the Brazilian Geotechnical community should be placed to regulate and standardize geotechnical information data transfer in digital format and to develop a national geotechnical investigation database. For that purpose, national quality standards and certification should be implemented. The development of Artificial Intelligence algorithms capable of “reading” and converting historic borehole reports into digital format can integrate such databases and form a live and dynamic geotechnical information network.

5 ACKNOWLEDGMENTS

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Hoover Dike USA – Experiences with the use of a digital twin in specialist civil engineering

J. Gottwald[1] and M. Daubner[2]

[1]IT-Project Manager, BAUER Spezialtiefbau GmbH, BAUER-Straße 1, 86529 Schrobenhausen; PH +49 8252 97-3639; email: jonas.gottwald@bauer.de
[2]Head of Digitalization, BAUER Spezialtiefbau GmbH, BAUER-Straße 1, 86529 Schrobenhausen; PH +49 8252 97-2377; email: marcus.daubner@bauer.de

ABSTRACT

Clewston, USA. Paper is outdated, the digital twin is reality!
This is how evaluations and production optimisations run in a matter of seconds in digital form. The customer is aware of that as well and that is the reason why the customer demand data management on the Herbert Hoover Dike contract. Production parameters are recorded every second by the CSM rig using various sensors and saved in a production file. In addition, all project-relevant data (reports, images, videos ...) are stored centrally and are made available for the customer on a daily basis. All project parties have the possibility to follow the process of the construction site on a digital replica and are able to start control measures for the execution or planning.

The production data are read into the relevant production data management system "b-project", processed, and stored in a standardised database. The production logs, quality checks or overlap calculations generated from this twin are created completely automatically by b-project. The overlaps between the individual constructed Cutter Soil Mix (CSM) elements are required by the customer in three different levels and every 10 feet (approx. 3 m).

No 2D/3D modelling software or other programs are required for the visualisation and calculations. The data and tools are implemented in the software. A data manager sets up the system on the construction site and raises an alarm if there are any quality defects. As a result, location-independent control measures can be initiated in the shortest possible time after the element has been manufactured. All data can be accessed worldwide. In this way, the efficiency of the measures initiated can be traced directly on the digital twin.

A 3D geographical information system (GIS) system opens another form of visualisation and documentation for the digital image of the project. This enables a uniform understanding of the project and visual monitoring of target/actual states.

Keywords: digital twin, digitalisation, GIS, data management system, BIM

1 INTRODUCTION

In the past few years, the execution of project sites has become more and more digital.

At present, there is no system available on the market which is tailor made for the requirements of large and complex specialist foundation projects and visualisation of the functionalities as demanded for worldwide application.

This was the reason why BAUER choose to take this direction to developing an own data management tool for special foundation projects, which can handle the production files of machines from the Bauer Machine GmbH (BMA). The newly created system has a degree of standardisation which creates a company standard for the future and allows for sufficient flexibility to cover multiple cases. Digitalisation has reached the special foundation industry. Not only construction companies have an ever- increasing necessity to extract data, especially production data, but also clients.

At the project Herbert Hoover Dike (HHD), a major project in the US, BAUER Spezialtiefbau GmbH (BAUER) developed a digital twin to satisfy the requirements of the contract and make forecasts about the current condition and future processes of the construction sites. This gives the opportunity to react to problems and adapt to increased productivity.

2 PROJECT DESCRIPTION

With a total area of 1,890 km², Lake Okeechobee is the largest freshwater lake in the US state of Florida. Despite its dimensions of about 56 km in length and 48 km in width, the lake is very shallow, with a 3 m depth on average and moreover has no major natural outflow.

As a protection against storm tides and floods, the existing dam was first extended in the 1920s. Additional canals, locks and dams were constructed.

Since 2007, the U.S. Army Corps of Engineers (USACE), a general command of the US Army, has been responsible for coordinating the refurbishment of the 225 km long dike around Lake Okeechobee.

BAUER Foundation Corp. was commissioned for the first time from 2007 to 2013 with the execution of cutter soil mixing sealing walls for the rehabilitation of the first sections of the dike. Due to the successful cooperation, the Bauer Foundation received further partial orders for the years 2011 to 2022, including for the sealing work on various sewer culverts.

In two constructions stages, they replace first the unsuitable organic materials by more than 35,000 predrill borings and fill it with non-organic backfill. Followed by an installation continuous and homogeneous cut-off wall. This wall has a cumulative total length of over 30 km and consists of more than 12000 panels by a depth of 20m and a width >= 60 cm.

3 PROJECT REQUIREMENTS

3.1 Digital requirements

The U.S. Army Corps of Engineers (USACE) operates and services approximately 740 dam structures, including the associated infrastructure in the USA. For the projects associated with the Herbert Hoover Dike Remediation, the USACE has listed items in their specifications which need to be provided in a digital format. The data exchange is a contractual obligation which has to be fulfilled by the Contractor. Here in Germany as well, more and more construction projects have “Client Information Requests” (AIA) incorporated in the tender documents. These AIA specify which kind of data and in which format it needs to be handed over to the Owner.

To comply with the owner’s request, the contractor compiles a Data Management Plan (DMP) which explains the concept how it is executed and transmitted. Such a plan was also compiled for the HHD project. It describes how the system is implemented and kept up-to-date. The DMP is considered to be a living document; it is updated in regular intervals, and if required, constantly discussed and agreed with the client.

BAUER had already developed an in-house platform to collect and analyse production data and save it in a structured way. In addition to the data transmission to the client, an internal analysing system was created during project execution, which supports the site with varied information and evaluations.

Further, additional tools were supplemented to broaden the project support, which lead to the development of a digital twin of the project that could be visualised.

These AIA specify which kind of information, when and in which format it needs to be handed over. Here, USACE determined two ways of doing so. A GIS-platform as well as data transmission via sFTP. Documents on quality assurance, but also information on construction progress, are examples of the required information. They are, to a large extent, transmitted in an automated way.

The following information has to be handed over in format.

GIS
- production report
- overlaps in different layers
- links to videos (VBs)/photos
- production data (start/stop)
- quantities (slurry)
- depths

Server storage
- pictures
- videos
- reports
- machine data
- test reports

3.2 Personal

The data manager named in the DMP is responsible for the whole data management on the construction site. The data manager also serves as a contact person for the customer in terms of all data-related topics.

On the part of the contractor, data manager takes care of the documentation and processing of the resulting data. This includes the design and accumulating AsBuilt data. The data must be secured, examined, and evaluated. Adaption is possible as well before they are made available to the customer in the GIS and file server in the form of drawings or protocols.

4 CENTRAL DATA RECORDING

The project collects data during the complete execution phase. The model is fed from all areas involved in the project. Data may come from CSM rig, design, but also from specialist departments. The data are archived centrally on the platform b-project developed by Bauer. The data may be retrieved in real time and analysed in various ways.

In order to make the data more reliable, a quality check is required before release, marked as “tested” and then passed on to other systems. By combining the differing sources, we gain advanced evaluation possibilities.

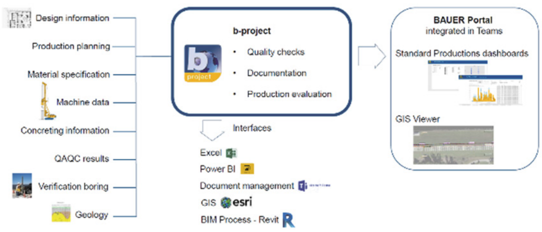

Figure 1. Interfaces of b-project

5 DIGITAL MODEL

Digital twins are the virtual representation of a physical object or system. The task is, for example, to understand, predict and optimise the performance of a plant or process. The aim is to achieve better business results and increase the overall performance of a company. Digital twins are made up of three components: a data model, algorithms and knowledge (Max-Ludwig Stadler, 2021).

On the b-project platform, the complete HHD project is available by means of datasets. They can be for example in 2D and 3D and are always compiled on the basis of the adjusted or latest information. The data may also be transmitted into data analysis systems and can be displayed by diagrams.

On a daily basis, reports are prepared in a Business-Intelligence-(BI)-solution and provided to the persons involved in the project via a portal.

By archiving the data in a system containing the same structure, they can be analysed transversely. In doing so, a wide range of possibilities opens up. The construction progress can, for example, be linked with the quality data. As well, specifications by the client can be compared with the construction data and automatically verified for compliance.

By archiving the data in a structured way, comparisons can be quickly drawn. The project can generate steps for a better and more effective construction sequence and check directly the effect of the relevant control measures.

6 PROJECT REPORTING

6.1 Tailor-made for recipient groups

As the construction process contains a great wealth of information, the digital twin, as well as user-group-oriented evaluations, can be implemented rather easily. Because of the digital twin, it is feasible to process the automatic analysis for various kinds of user groups and to provide it in an easily accessible environment. The data is analysed – if necessary – several times a day.

To make sure that the users widely accept the tool, its usage has to be kept simple. For Bauer's digitalisation strategy, it was decided to use Microsoft Teams by default. That means that for each project a separate environment is created in Teams where the "BAUERdigital Portal" is incorporated. Here, information can be saved via a tiles navigation. These may be linked to applications, such as the GIS-Portal as used in the HHD project or to the reporting system.

In the last years, BAUER has generated standard reports on each major project which uses b-project. They are respectively available for all construction methods, as carried out by Bauer and tailor-made for three main recipient groups. The latter ones are the client level, the management level, the site management, and the specialist departments of the project, as well as the site management on the construction field.

Each project site is able to grant rights for the system on its own. To do so, a simple system was developed. The relevant "owners" of the teams are in a position to administer who to grant which rights and when, i.e. they do not need any support by the IT department in this respect.

Management and client

- Simple visual display, giving an overview of construction progress
- Target/actual overview by means of speedometer

Project management

- S-curves with target/actual display
- Forecast on completion date
- Display of delays

Site staff

- Detailed information on production
- Quantity information on consumable material
- Information on deviations during production

Figure 2. Example Business-Intelligence evaluations of the project

6.2 Consistent quality despite of high number of elements

The more elements there are at a project, the more effort has to be taken for quality assurance, documentation, and reporting. The platform b-project as used by BAUER reduces this effort. The number of elements is irrelevant here, which represents one of the greatest advantages of introducing a standard system. The reason is that logic of creation such documents is implemented in the system. As soon as the data has been entered and checked, the creation can be completed without manual intervention.

Quality and construction progress are documented according to the time interval as required by the project. Apart from the documentation, the system also supports optimising the construction sequence by its timely reporting system. Here, data from the current production may consistently be compared with the target requirements of the project. If applicable, countermeasures can be taken. Control measures are directly displayed, and further corrections are possible.

The overall quality of project documentation as well increases that way. It is constant and comparable throughout the complete execution period. Changes in the reports are applied for the complete construction period and are not only visible starting from the day when the change was made.

Moreover, mistakes in the planning phase or the construction sequence can be quickly and easily identified and directly rectified. Thus, there is less potential for subsequent defects. As well, optimisation of elements can be identified, and it is also possible to develop potential for improvements for subsequent projects. The system immediately compares the geometry as planned with the one of the components installed in 3D view, thus enabling a visual check of the geometry. This comparison can be done daily. The information on the elements installed is automatically displayed in the CSM rig.

7 VISUALISATION

7.1 GIS-System

In a geo-information system, data of various origins are linked with the relevant location in the model. In this way, it is possible to visually locate project-relevant documents and information. The attached documents, videos, and photos can be opened and downloaded via a web browser at any time.

USACE required during the tender phase that data be provided on a GIS portal. Here, it was specified in the contract which data was to be retrievable in GIS and in which period of time they have to be available after completion of the individual elements.

An interface was programmed which ensured that the data is transmitted directly from b-project into the GIS-system, which is spliced in two different working layers.

7.1.1 Plan view

Is the top view of the georeferenced online map. Each element constructed can be selected manually and visualizes the general production information. Here, the client requires submittal amongst other information of the respective position data, start and stop times, the slurry quantity as consumed as well as the final depth of the individual element.

The required documents are automatically generated in b-project and then archived in a uniform structure and under a pre-defined glossary in the GIS system. The following information was required by USACE and implemented accordingly.

The production reports of an element have to be linked with the twin displayed in the GIS-model. This means that each constructed element has a virtual counterpart; the relevant construction report is attached to this counterpart in the model.

For each element, overlap drawings have to be created. They should to be generated for the depths as specified in 2D-view. The drawings, photos, and videos are also linked to the relevant elements and placed at the relevant position in the GIS where they originated.

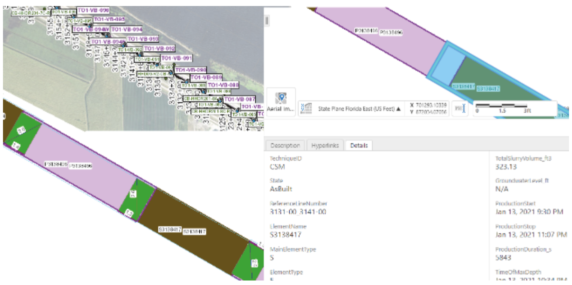

Figure 3. GIS plan view

7.1.2 Profile view

Exploration borings, verification boring of injected material and QAQC test results are paramount in this view.

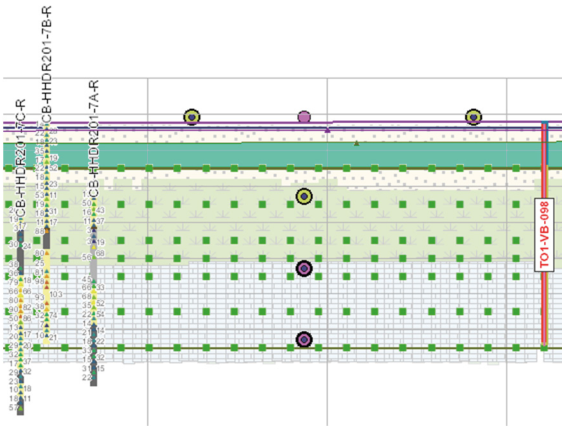

Figure 4. GIS profile view

In Figure 4, the circles demonstrate QC results of tested material. Circles are placed on different heights, which stand for the place/height of sampling. Test results and testing documents were displayed as well.

Before the project was starting, BAUER executed on different stations exploration borings to identify the existing geology. Based on these results, the construction site profile is optical classified in different soil layers. All relevant documents are linked in the GIS. One of the biggest parts is the demonstration of the verification boring of the As-Built elements. Videos which are recorded after the boring were attached on the correct spot and can be easily accessed by the client.

7.2 Documents and Drawings

Due to the client's requirements, various further documents and drawings in 2D are also necessary. These are mainly displayed in pdf-format. In this case, we are talking about production reports, overlap calculations, construction reports and overlap drawings.

The data as needed for these reports are made available in a structured way and are archived in the b-project database. The resulting drawings and calculations may be generated at the touch of a button. Expert knowledge on CAD or a similar drawings software is not required.

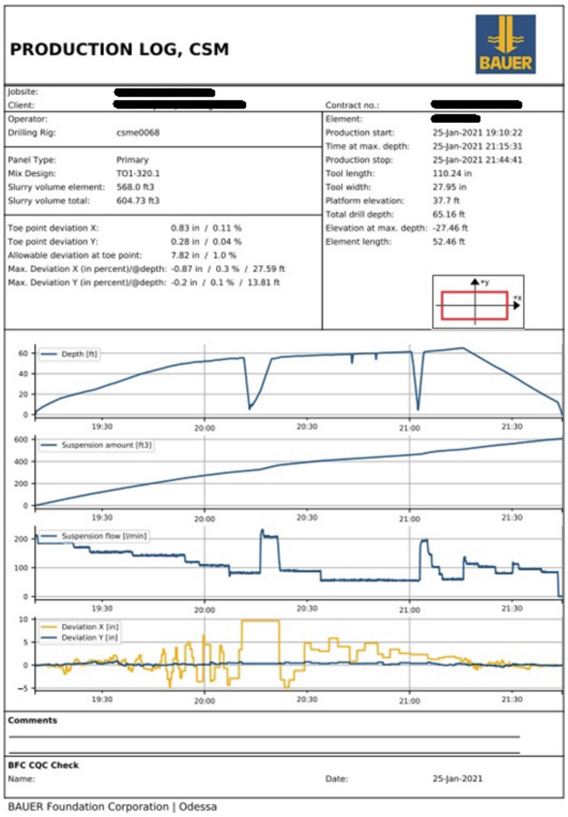

Figure 5 Production protocol CSM

Overlap drawings may be generated at any time and variably. In Figure 6, overlaps of a secondary element with both primary elements are displayed. The required cut-off heights (Top, Mid, Bot) were pre-defined by the client. Position data and deviation data of the individual elements are the basis of these drawings as saved in the machine data and archived in the system database.

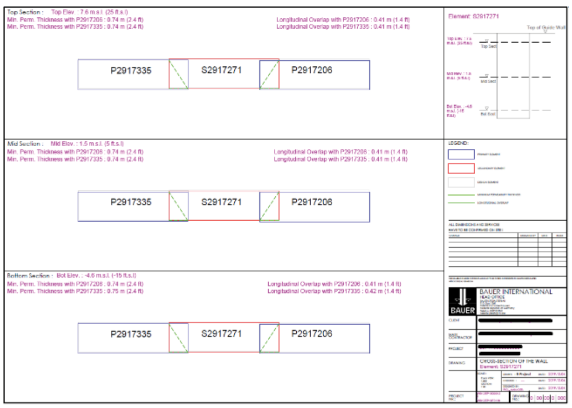

Figure 6. Automated overcut calculation drawing

Everything is provided, taking minimum effort and almost in real time. The document information record, as required by the client, can be retrieved at any time and is archived in a structured way. An Add-on was created to compile the documents, which displays the necessary documents and information when needed.

7.3 As-Built in Revit

The design data are recorded in b-project. They are available in list format and contain all information as required for the production. These data may also be used to generate a design model in the modelling software Revit.

b-project processes and archives in a structured way, the production data as collected by the CSM rig, the so-called “As-Built” information. A 3D-model can be generated based on these data via the relevant interface in Revit. The recorded geometrical data are processed in Revit using a dynamo script and an As-Built model of the elements is compiled. They are used to facilitate comparisons and form the basis for data checks. Here, it is easy to visually spot if the elements were installed at the correct position. As well, it is also quite simple to check if the targeted final depth was reached or if it was exceeded.

8 CONCLUSION

As this tool was developed in-house, it is flexible and can be used adjusted to any projects with little effort. Other features can also be implemented within a relatively short time.

It is also possible to link it to external sources. During the development phase, it was important to generate a sufficient number of interfaces to enable the connection of other systems, if applicable also external partners.

The advantages of using a data management system like b-project are obvious. The system can be used by everybody after a short training; no expert knowledge for drawings, calculations or similar are necessary. As described, a digital twin does not always have to be a graphical interface, but makes life easier. The representation of the resulting data is very clear. A picture is worth than thousands words. It increases understanding and facilities discussion between the contracting parties. The data of the digital twin is stored structured and can be used at any time for necessary evaluations. E.g. between similar projects based on the same geology can be obtained and included in future tenders.

The database of the digital twin is the basis for improvements in construction processes or further development of the machines.

In addition, there is no risk that, in contrast to other types of documentation, the data will be lost due to the termination of employment. The central data storage of the database protects them for later application.

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SESSION 3

ADVANCED NUMERICAL MODELLING

Photo courtesy of Esmaeil Pourshafiei (TonkinTaylor)

Helical Pile Installation for Offshore Renewable Energy Exploration in Clay Seabed

Y. Hu[1], M. Zhang[2] and S. N. Ullah[3]

[1]Professor in Geotechnical Engineering, School of Engineering, The University of Western Australia, Email: yuxia.hu@uwa.edu.au
[2]PhD Candidate, School of Engineering, The University of Western Australia, Email: meili.zhang@research.uwa.edu.au
[3]Senior Lecturer, School of Engineering and Technology, Central Queensland University, Email: s.ullah@cqu.edu.au

ABSTRACT

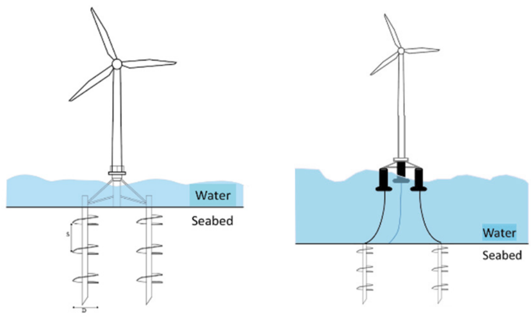
Offshore wind and wave energy exploration are moving from shallow waters with fixed foundations to deep waters with floating devices. Helical pile has the potential to be used as both shallow water foundations and deep water anchors due to its 'quiet' installation and environmental friendliness to marine living systems. Although helical pile and/or anchor have been used extensively for onshore applications, their offshore applications need larger diameters and longer shaft than the onshore counterparts hence pose significant installation challenges. This paper presents the current studies on helical pile installation process in clay seabed. The installation of helical pile in uniform and normally consolidated clay have been studied physically in centrifuge and numerically using large deformation finite element (LDFE) analyses. Both installation torque and installation force (or crown force) were studied under different pile-soil friction coefficients and different helical pile advancement ratios (AR: ratio of pile penetration to helix pitch). Soil flow mechanisms under different ARs can explain the development of required toque and crown force during torsional pile installation. The installation torque and installation force are a function of AR and can be designed based on the capacity of installation equipment.

Keywords: helical anchor, clay seabed, offshore energy, numerical analysis, installation, capacity

1 INTRODUCTION

Offshore renewable energy exploration, such as wind and wave energy, is moving from shallow waters with fixed foundations to deep waters with floating devices (Figure 1). To secure these fixed and floating devices, the design of economical and environmentally friendly foundation/anchor systems is vitally important. Among all different foundation and anchor options, such as monopile, drag anchor, plate anchor and pile anchor, helical pile is attracting more and more attention due to its quieter installation with less noise emission into the ocean environment as the installation noise may harm marine lives.

Helical pile has been applied to support marine and onshore structures for a few decades. Its early application was offshore in shallow waters in the 1950s for developments in marine environment (Feld, 1953). However, over the last few decades, the development of helical piles/anchors application was largely to support onshore structures, such as transmission towers, billboard lights and sign foundations (Lutenegger, 2017). The onshore helical piles/anchors generally have small shaft diameters in the range of 0.0138 m ~ 0.51 m. The helix diameter is in the range of 0.15 m ~ 0.9 m. Table 1 shows a few examples of onshore helical piles in soils with fine particles. Even to support a large residential building with 11-storeys, the helical piles had a shaft diameter of 0.05 m and their helix diameters varied from 0.2 m to 0.3 m. (Vito & Cook, 2011).



(a) Helical pile (b) Helical anchor

Figure 1. Offshore wind turbine with helical pile and anchor foundations

Table 1: Samples of recent studies of onshore helical piles in silt and clay

Researcher	Year	Location	Ground Condition	Shaft Diameter d (m)	Helix Diameter D (m)
Sakr	2011	Field tests, Alberta, Canada	Sand, stiff glacial till	0.32-0.51	0.76-1.02
Vito & Cook	2011	11-storey building, Ontario, Canada	Sandy silt, silt	0.051	0.2-0.3
Weech & Howie	2012	Field tests, British Columbia, Canada	Peat, deltaic clayey silt, marine silty clay	0.089	0.356
Papadopoulou et al.	2014	FE, Plaxis 2D	Sand and clay	0.076	0.22
Alwalan & Naggar	2020	FE, Plaxis 2D	Sand and clay	0.324	0.61

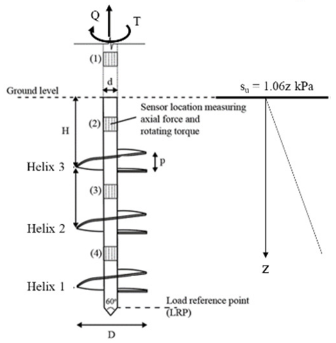
The helical piles designed for offshore renewable energy explorations need larger shaft and helix diameters than their onshore applications. This is because the offshore structures are experiencing larger environmental loadings, such as wind, wave and current, and structural loading as offshore structures are larger and heavier than the onshore ones. Due to the large size of offshore helical piles, the torsional installation offshore can be challenging.

This paper presents some development in the helical pile research in clay seabed. The studies include physical model tests using centrifuge and numerical analysis using LDFE. The required torque and crown force were investigated for optimum installation designs against pile advancement ratio.

2 CENTRIFUGE TESTS

2.1 Model pile - 1

The helical pile in normally consolidated (NC) clay was tested in centrifuge under 50g (Ullah et al., 2019). The model pile has three helices (Figure 1) and the installation torque and crown force were measured in four locations to investigate the distribution of each helix during installation. The pile was installed with a constant crown force. Its advancement ratio was estimated as AR = 1.0, which means the pile penetrates one pitch per revolution. Soil undrained shear strength profile, pile dimensions and sensor locations are also shown in Figure 2.



H: top helix embedment

Prototype dimensions:
D = 2 m, helix diameter
d = 0.5 m, shaft diameter
p = 0.42 m, helix pitch
S = 2 m, helix spacing

Sensors measurements:
Sensors at locations (1), (2), (3) and (4) are measuring installation torque and crown force T1, T2, T3, T4 and Q1, Q2, Q3 and Q4 respectively

Figure 2. Model pile - 1 in centrifuge test

2.2 Centrifuge test results

The pile was installed in NC clay. The normalised installation responses are displayed in Figure 3. As s_u = 1.06z kPa, the average s_{ua} = 4.24 kPa over the pile penetration of 8 m is used in normalisation. Positive Q and T are defined in the figure. The penetration depth is recorded for the reference point (RP). The crown forces are compressive from all sensors. It can be seen that the total installation force has Q1 ≈ Q2 in Figure 3a, as there is no extra helix between these points (1) and (2). The shaft between these two points has minimum effect due to its small diameter and soft soil near the surface. The crown force of Q4 from Helix 1 is developing tensile force with penetration depth. This means that, with AR = 1.0, the rotation of the bottom helix can generate tensile force after it is fully embedded in soil. This phenomenon can also be observed when Helix 2 is fully embedded in soil. This is interesting, as this phenomenon shows less reliance on the crown force during installation.

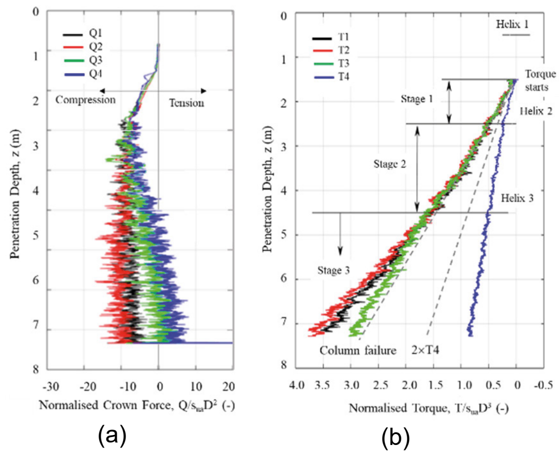


Figure 3. Individual helix effects on installation torque and force of model pile-1 under constant crown force

Figure 3b shows the development of installation torque with pile penetration. The torque T4 measures the torque resistance from Helix 1 and it increases linearly with penetration depth as the soil strength increases linearly with soil depth. However, as Helix 2 enters the soil, the total torque from Helices 1&2 (i.e. T3) is more than double the T4 (i.e. T3 > 2×T4). This means that Helices 1&2 are not working as two individual helices. This is consistent with other research findings that the spacing between two helices needs to be S/D ≥ 3 (Alwalan & Naggar, 2020) for individual helix behaviours. The helix spacing here is S/D = 1.0, hence a soil column failure between the bottom two helices is expected. The estimated torque resistance from the column failure matches the measured T3 fairly well. The T1 & T2 are slightly higher than T3, as the Helix 3 embedded in top soil with low strength. At the same time, the torque resistance from the shaft between sensor points (1) and (2) is minimum, due to small shaft and soft surface soil.

3 LARGE DEFORMATION FE ANALYSIS

3.1 Model pile - 2

To study a single helix contribution to the installation torque and crown force, a pile with a single helix is analysed in a uniform clay. Figure 4 shows the helical pile dimensions and soil strength profile. The definitions of all variables are listed in Figure 4 for easy reference.

3.2 Large deformation FE analysis

The torsional installation of the helical pile is simulated using 3D large deformation finite element (LDFE) analysis with coupled Eulerian Lagrangian (CEL) method in ABAQUS (2011). The FE model set-up is depicted in Figure 3. The pile shaft and helix plate are modelled as a rigid Lagrangian body and the soil domain is discretised using 8-node brick elements

(EC3D8R). The mesh is refined around the helical shaft and helix plate with minimum element size of D/40. The soil domain is set large enough to avoid boundary effect (see Figure 5).

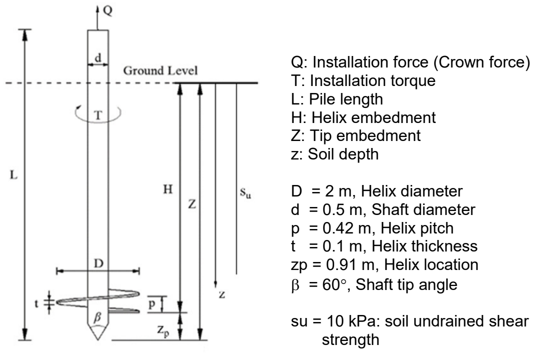


Figure 4. Model pile – 2 in LDFE analysis

The soil is modelled as a medium soft uniform clay with its undrained shear strength of s_u = 10 kPa. The soil Young's modulus is set as medium stiff clay of E_s/s_u = 500, which is commonly used to simulate offshore clays (Hu et al., 1999). The Poisson ratio of ν_s = 0.49 and fiction and dilation angles of $\varphi = \psi = 0$ are set up for undrained analysis. Soil submerged density is ρ_s = 700 kg/m^3.

The helical pile is simulated as a weightless rigid body. The interface friction between soil and pile varies as α = 0 ~ 1.0, where α = 0 is fully smooth and α = 1.0 is fully rough.

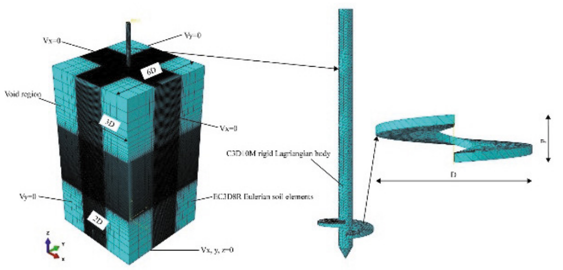


Figure 5. 3D LDFE model of model pile - 2

3.3 LDFE/CEL model validation

The LDFE/CEL model is validated against centrifuge test data using model pile – 1 in Figure 2 (Ullah et al., 2022). The LDFE/CEL results are compared with centrifuge test data of total resistant torque (i.e. T1 in Figure 3). It can be seen that the numerical results agree well with the centrifuge test data. The slight over estimations of the numerical result may be due to the strain-softening effect of centrifuge clay (i.e. clay sensitivity S_t = 3, Ullah & Hu, 2020), which is not considered in the numerical analysis. However, the torque from the numerical analysis shows the same trend as that from the centrifuge test.

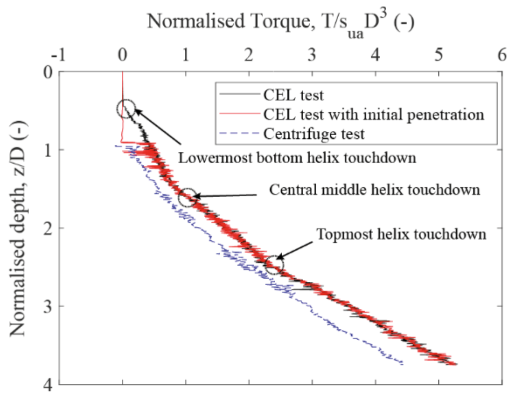


Figure 6. LDFE/CEL mode validation of model pile – 1 in centrifuge test

3.4 Numerical analysis result

In this study, the effects of helical pile friction and pile advancement ratio on the installation torque and crown force are studied. The advancement ratio (AR) of the helical pile is defined as

$$AR = \text{vertical displacement per revolution}/p \quad (1)$$

where AR = 1.0 is neutral rotation (i.e. one pitch vertical displacement per revolution); AR < 1 is over rotation (i.e. fast rotation/slow penetration) and AR > 1 is under rotation (i.e. slow rotation/fast penetration).

Pile friction effect

The pile friction effect at AR = 1.0 and s_u = 10 kPa is displayed in Figure 7. It can be seen that, under neutral rotation, the crown force is compressive. With increasing penetration depth, the compressive crown force is decreasing in magnitude. This means that, at AR = 1.0, the helix is generating tensile force, which is similar to the observations in the centrifuge test (see axial force Q4 in Figure 3a). The pile friction has minimum effect on the crown force at α = 0~0.45 (Figure 7a), where common friction coefficient lies for steel piles.

However, the pile friction has profound effect on installation torque. The torque increases linearly with increasing friction coefficient α (Zhang et al., 2022). A formula was developed by Zhang et al. (2022) to calculate the torque as a function of z/D and α.

It is apparent that the required installation torque and crown force can be minimised when the pile is relatively smooth.

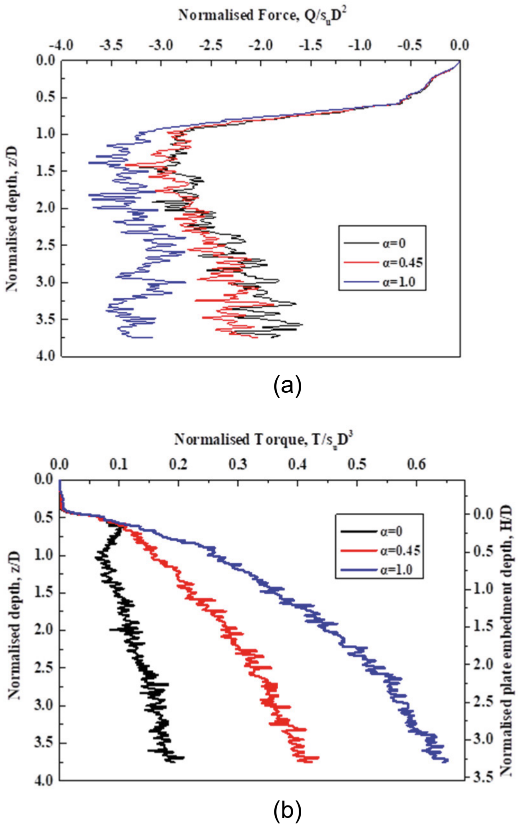


Figure 7. Pile friction effect (AR = 1.0, s_u = 10 kPa)

Advancement ratio (AR) effect

The effect of pile advancement ratio is depicted in Figure 8. It is interesting to see that, when the helical pile is over rotating (i.e. AR < 1.0, fast rotation/slow penetration), the crown force changes from compression to tension, as it is also observed in centrifuge tests. (i.e. Q4 in Figure 3a). On the contrary, when the pile is under rotation (i.e. AR > 1.0, the pile is pushed into soil with faster penetration than rotation), hence the crown force increases with increasing pile penetration depth until a constant crown force is reached at z/D = 2.5. Thus, it is desirable to install the helical pile with AR ≤ 1.0 (i.e. over rotation).

However, with AR ≤ 1.0, the required installation torque becomes higher under neutral and over rotation. Thus, the crown force can be reduced by increasing installation torque.

By correlating the crown force graph to the torque graph in Figure 8, there is a potential to provide an optimal installation design, based on the installation equipment capacity. Figure 9 shows the preliminary design chart based on pile installation AR. The required crown force and torque during helical pile installation are data in Figure 8 at z/D = 3.5. <u>*Example 1*</u>: minimising required crown force (i.e. Q ~ 0.0, A→B). From Q = 0.0 at point A, the required is AR = 0.84, where T/s_uD^3 = 0.45 at point B. Thus, the required installation torque can be estimated. <u>*Example 2*</u>: under allowable installation torque of T/s_uD^3 = 0.3, C→D. From the allowable T/s_uD^3 at point C, the required is AR = 1.19, where the required crown force is Q/s_uD^2 = 4.2 at point D. Then the required crown force can be calculated.

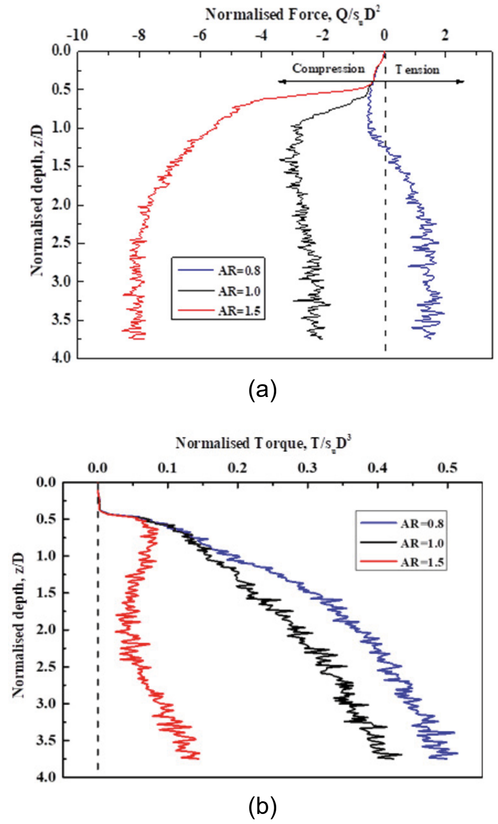


Figure 8. Pile advancement ratio effect (α = 0.45)

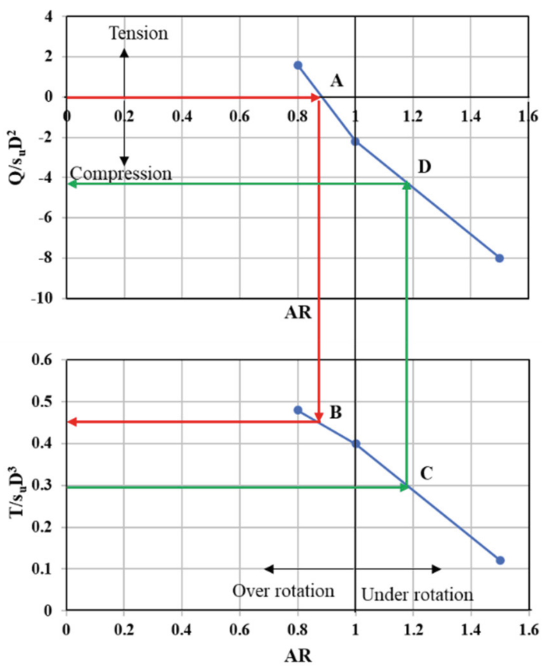
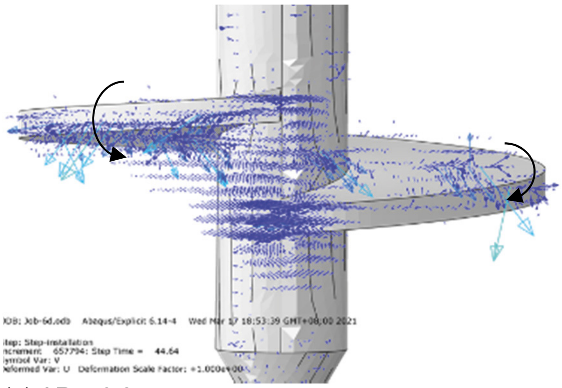


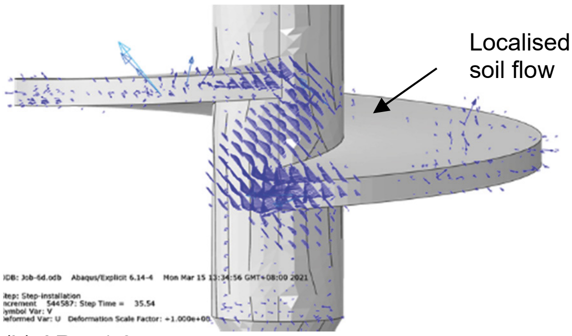
Figure 9. Required crown force and torque at α = 0.45, z/D = 3.5

3.5 Soil flow mechanisms at various AR

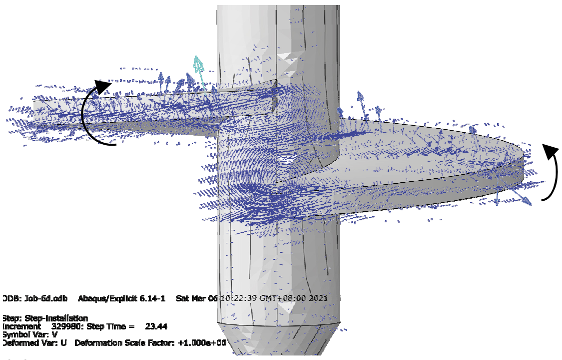
The soil flow mechanisms during helical pile installation in a uniform clay are depicted in Figure 10. It can be seen that, under neutral rotation of AR = 1.0 in Figure 10b, the soil flow is localised around the top and bottom ends of the pitch. However, with over rotation of AR = 0.8 (i.e. fast rotation/slow penetration), the soil around the helix brim is flowing from the top of the helix downwards. This indicates a tensile force is generated, since a horizontal plate subjected to upwards tensile force can induce soil flowing from top to bottom of the plate around the brim. On the other hands, with under rotation of AR = 1.5 (i.e. slow rotation/fast penetration), the soil around the brim is flowing from the bottom of the helix upwards. This is a typical soil flow mechanism when a horizontal plate penetrates into clays with a compressive force (Song et al., 2008).



(a) AR = 0.8



(b) AR = 1.0



(c) AR = 1.5

Figure 10. Soil flow mechanisms during helical pile installation with α = 0.45 and s_u = 10 kPa

4 CONCLUSION

This paper presents current studies on the helical pile installation in clay seabeds in centrifuge test and by LDFE/CEL numerical analysis. In the centrifuge test, a triple helices pile was installed in a NC clay. In the numerical analysis, a single helix pile was installed in a uniform clay, after the numerical method was validated against the centrifuge test data.

In the centrifuge test, it was found that the bottom two helices played a major role in crown force and installation torque development. A clear tensile force was generated by the bottom two helices. The bottom two helices were acting together to form a column failure to resist torque.

In the numerical analysis, the pile friction had minimum effect on the crown force when α = 0~0.45. However, the pile friction had a strong effect on installation torque, where the installation torque increased linearly with the friction coefficient.

The pile advancement ratio played a major role in the development of crown force and installation torque during the rotational installation of a helical pile. Under rotation was not recommended due to high crown force requirement. Neutral and over rotation can be designed based on the installation equipment capacity.

More analyses are underway to establish a full design chart to guide helical pile installations in clay.

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Comparing numerical modelling finite element results with full scale instrumented pile response in weakly to moderately cemented soil

B. S. Riyat[1], MIEAust CPEng, S. W. Lee[2], FICE (UK), and S. Y. Fok[3]

[1]Senior Associate Geotechnical Engineer, WSP Golder, Building 7, Botanica corporate park, 57/588 Swan ST, Richmond VIC 3121; PH (+61) 3 8862 3684; email: bhavik.riyat@wsp.com
[2]Former Principal, WSP Golder, 7th Floor, One Kowloon, 1 Wang Yuen Road, Kowloon Bay, Hong Kong, PH (+852) 6036 3679; email: geoswlee@gmail.com
[3]Assistant Engineer, WSP Golder, 7th Floor, One Kowloon, 1 Wang Yuen Road, Kowloon Bay, Hong Kong, PH (+852) 66761060; email: sunny.fok@wsp.com

ABSTRACT

Design of drilled and grouted piles in cemented soils remains one of the challenging geotechnical problems mainly due to the variable degree of cementation in the carbonate soil and poses uncertainties to the design of foundations for offshore wind farms structures. Carbonate soils encountered in offshore Australia have embedded layers of loose and compressible cemented soft rocks to well-cemented calcarenites. The purpose of this paper is to undertake finite element analyses using PLAXIS software to quantify pile-soil interaction and predict pile response and load distribution along the pile length under various loading conditions. The results from full scale instrumented piles in weak to moderately cemented soils are presented to draw the comparison between experimental and numerical results. The results from in situ (cone penetration test, CPT) and advanced laboratory testing (constant normal stiffness, CNS) on variable cemented soils that replicate the loading imposed by waves on offshore wind farm structure are used to demonstrate strength degradation in carbonate materials during cyclic loading.

Keywords: Numerical analysis, full scale instrumented pile, PLAXIS, pile-soil interaction

1 INTRODUCTION

Typical structures like offshore wind farms, high-rise buildings and cut and cover metro stations require an adequate foundation system to satisfy ultimate and serviceability limit conditions against uplift forces. In the offshore wind structure conditions, environmental (i.e., wave, currents and wind) and loads produced due to vibration caused by the blade shadowing effect (referred as 2P/3P). In onshore structures, mainly high ground water pressure acting on the base slab of cut and cover structure contribute to uplift forces.

Mainly two components, one weight of the super structure and second is the shaft friction acting along the pile length resist the uplift force. It is important to use more established reliable correlation to estimate shaft capacity of piles, since the shaft capacity depends on the installation type of pile and stratigraphy. In offshore environment, calcareous soils varying from soft uncemented silts to dense well cemented calcarenite and limestone, as encountered in the North West Shelf of Western Australia, pose significant challenges in design. These challenges are further exacerbated by the cost and difficultly of performing full scale load tests in offshore. Drilled and grouted piles are preferred over driven piles in calcareous sediments due to the well documented low shaft frictions that develop on driven piles. However, Joer and Randolph (1994) show that the shaft friction on drilled and grouted piles exhibits significant post-peak strain softening and degrades rapidly with cycling. This is due to a loss of cementation/bonding with the application of shear strains coupled by contractant response of the materials under cyclic loads that leads to reduction of radial stresses on the pile shaft and consequent loss of shaft friction capacity.

The traditional approach for estimating the shaft friction capacity of drilled and grouted piles in moderately cemented calcareous soils is to use empirical correlations that relate shaft friction with the unconfined compressive strength (UCS) e.g. Abbs and Needham (1985). However, sampling of weak rocks for UCS testing is problematic (and often unfeasible) and recent investigations using instrumented piles have improved the understanding of the factors governing the shaft capacity of drilled and grouted piles in weak rock. Therefore, there is a preference for direct correlations with the Cone Penetration Tests (CPT) end resistance (q_c) e.g., Randolph et al. (1996), Lehane (2011) and Riyat and Lehane (2018).

This paper presents the results obtained from onshore in-situ, full-scale tests carried out in 2010 on instrumented drilled and grouted piles at a calcareous sandstone site at Pinjar, Western Australia. The results are compared with the predicted pile response from a series of two dimensional (2D), axisymmetric finite element (FE) parametric analyses using PLAXIS. The weak rock was modelled using a non-linear elasto-plastic constitutive model, Hardening Soil (HS) (Schanz et al., 1999).

2 SITE SOIL/WEAK ROCK CHARATERIZATION

2.1 In situ testing

The site selected for pile load testing is a limestone quarry in Pinjar located about 25 km north of Perth, Western Australia. The weak rock is described as “medium grade limestone” which is a part of the Tamala Limestone formation. Visual inspection on site from test pits indicates the presence of cemented bands interlayered with weakly and very weakly cemented material (Guo and Lehane 2016). The carbonate contents measured on the cored samples by Lehane (2011) ranged between 40% and 55% and by Guo and Lehane (2016) were typically 40%±25%, indicating sediment deposits classified as calcareous sandstone according to Clark and Walker (1997). The sample core recovery was poor and ranged between 50% and 75%. Core samples recovered during the site investigation are shown in Figure 1, which indicate deposits comprising cobbles, thin cemented layers of sand matrix with varying cementation from uncemented to weakly cemented.

Figure 1. Photo of typical core recovered

The point load Index (Is50) values for the weakly cemented sample documented in Lehane (2011) were typically 0.15±0.05 MPa and were consistent with the range of Is50 values obtained by Guo and Lehane (2016) on samples recovered from the same site.

A total of eight CPTs were carried out across the site, including three tests in the vicinity of each pile location. The CPT profile presented in Figure 2 shows the spatial variability in end cone resistance (q_c) measured across the site with cemented layers existing at varying depths. Typical q_c values range from 15 MPa to 50 MPa with a few lenses showing q_c in excess of 80 MPa and less than 10 MPa. Few CPTs were terminated at shallow depths of less than 3 m. The variation in the q_c values attributes to the range of cementation with larger q_c values are attributed to high levels of cementation, while lenses with q_c values less than 10 MPa are largely uncemented. Friction ratios (F_r) generally lie between 0.2% and 1.0%, but are occasionally higher in the silty layers, for which q_c values are less than 2 MPa.

In typical offshore conditions, carbonate soils comprise greater than 90% CaCo3 and exhibit much softer response of q_c with thin layers of cementation compared to the onshore response of q_c shown in Figure 2.

Figure 3 shows CPT correlations to determine the soil behaviour type (Robertson, 1990). The majority of the pairs of normalised cone resistance (Q_t) and Friction ratio (F_r) fall within zone 6 and zone 7, indicating sediments comprising clean sands to silty sands and gravely sand to sand, respectively. The results also represent the drained penetration, and increase in cementation is evident in the soil behaviour type plot.

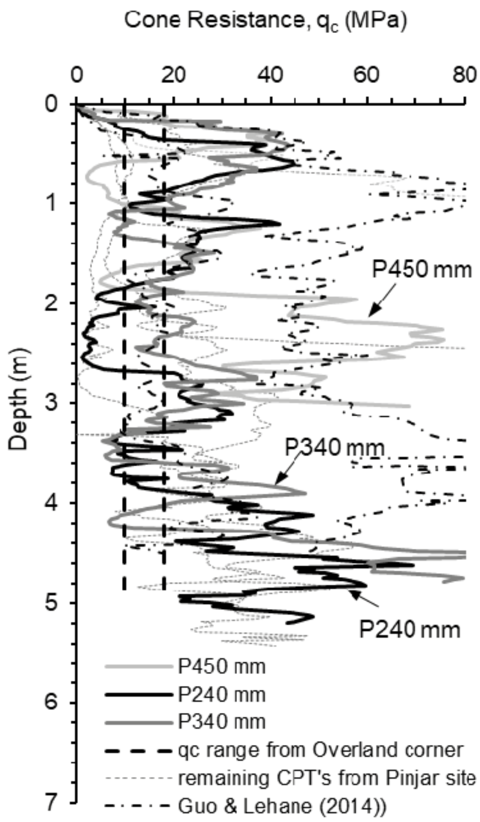

Figure 2. q_c profile of CPT tests from Pinjar site

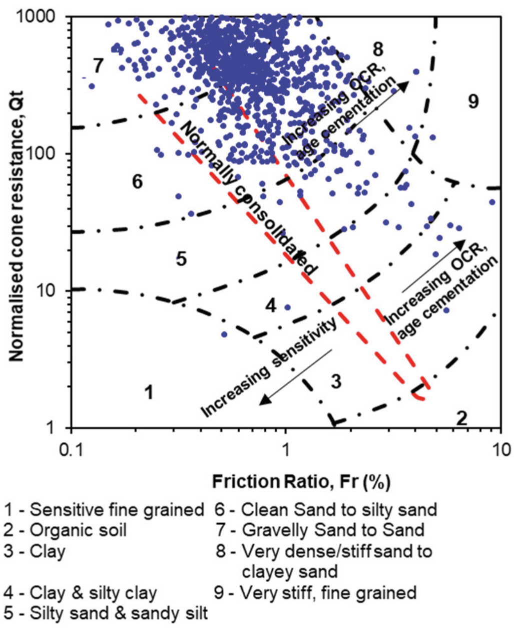

Figure 3. Soil behaviour type classification chart.

2.2 Advanced laboratory testing

Constant normal stiffness (CNS) direct shear tests that replicate the constraint of the surrounding formation around pile surface were carried out to assess the monotonic and cyclic shear response of soil for the design of axially loaded piles, particularly drilled and grouted piles (Johnston et al. 1987). The dilative or contractant response that occurs at the pile soil interface has a significant effect on the confining stress and hence on peak shaft capacity. In the CNS direct shear test, the vertical load is adjusted to maintain normal stiffness ($K_n = \Delta\sigma_v/\Delta h$) during shearing. The stiffness K_n represents the cavity expansion stiffness of the surrounding rock mass, therefore $K_n = 4G/d_{pile}$, where G is the shear modulus of the weak rock and d_{pile} the pile diameter.

The stress paths from the CNS tests on samples recovered from the site are presented in Figure 4. The samples were first consolidated to a vertical effective stress of 50 kPa before brought to failure through monotonic shearing, maintaining constant stiffness of 400 kPa/mm followed by cyclic loading. The samples indicated a stiff initial response reflection of cementation, with a gradual increase in strength until the shear stress reached the peak. Strain softening response was observed in most of the samples. Higher strength of the sample at depth 2.75 m is possibly due to the dilative response during shearing. From the stress path plot, peak friction angle (φ') varies between 39° and 44°. The findings of the cyclic loading on the CNS test samples are well documented in Lehane (2011) and Riyat and Lehane (2018). In general, large strength reductions were experienced in the initial cycles and the rate of strength degradation reduced with increasing cycles.

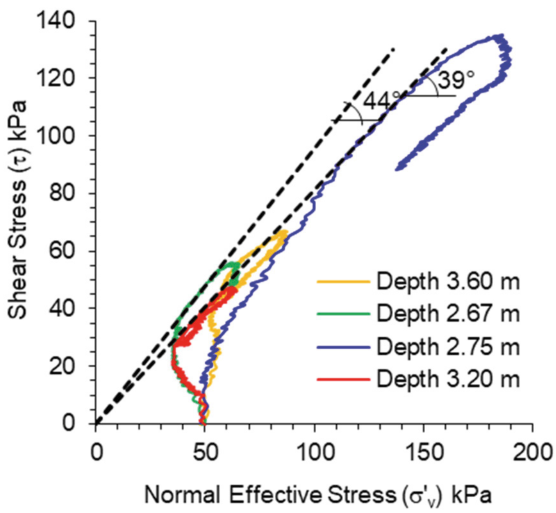

Figure 4. Stress path of monotonic CNS test results

The overall response of strength reduction is well captured in the CNS cyclic test, which is partially due to the shearing nature of the frictional materials but mainly unloading of normal stress with cycles. The volume loss of the sample which can be caused by loss of material (predominant in cemented samples) during cyclic shearing or by sample contraction significantly influences the normal stress. However, loss material was not investigated at the time of testing the samples. The sensitivity of the normal stress on a pile to change in volume in the adjacent soil is captured in DeJong et al. (2003). Randolph et al. (1998) also noted a large reduction in friction under displacement limited to 2-way cyclic loading.

3 FULL SCALE INSTRUMENTED PILE TEST

5 m deep bores for the test piles were drilled in September 2009 using 225 mm, 325 mm and 440 mm diameter augers. Experience with drilling using these augers in similar ground conditions indicated that the final bore diameters formed were 240 mm, 340 mm and 450mm. The holes were tremie grouted with 30 MPa (28-day cube strength) grout before insertion of the steel tubes. These tubes were beaded to ensure good shear transfer and instrumented with 20 quarter bridge strain gauges with two gauges at 10 levels. Tension tests on the piles were performed about 1 month after installation using the test setup described in Lehane (2011). Cyclic testing followed monotonic loading to failure.

3.1 Pile test results

The axial forces for 240 mm pile diameter derived from the axial strains that developed along the pile length for each increment of load applied at the pile head are shown in Figure 8. The unit shaft friction was estimated by taking the difference in axial forces at two subsequent strain gauge levels and divided by the pile surface area. The peak unit shaft friction from the strain gauges data is mobilised between 1.5 m and 3.0 m depth. The *β* value, which is the ratio of average CPT q_c to the peak shear stress, is typically 90±10 for the 240 mm and 340 mm piles (Lehane, 2011). The 450 mm pile shows a higher β value compared to the smaller diameter piles. The trend is consistent with the results shown by Rollins et al. (2005) for bored piles in sand and by Lehane et al. (2005), where centrifuge results showed the reduction in effective normal stress (smaller dilation) with increased diameter and stress at deeper depths which leads to a corresponding increase in the β values. This also explains the reasoning of micro-piles exhibiting higher shaft friction that corresponds to a lower *β* .

The load-displacement and cyclic responses observed at the pile heads are well captured and presented in Riyat and Lehane (2018). The peak tensile loads of 1280 kN, 1530 kN and 950 kN were observed for the 240 mm, 340 mm and 450 mm diameter piles, respectively. The significantly lower capacity indicated by the 450 mm pile highlights the variability in the ground conditions at the test site. The 450 mm pile also shows the highest level of post-peak brittleness, indicating ground conditions at this location differed from those at the other two locations which give quite similar peak and ultimate shaft shear stresses.

Comparing the cyclic response of the 240 mm and 340 mm piles, it is observed that higher amplitudes of cyclic load lead to larger cyclic displacement accumulations (Riyat and Lehane, 2018). However, more rapid accumulation of cyclic displacement is observed for the 450 mm pile with similar cyclic load amplitude compared to the 240 mm pile, illustrating the difference in ground conditions at the two respective locations and possibly also reflecting scale effects. The post-cyclic monotonic capacities for the 240 mm, 340 mm and 450 mm piles are 86%, 78% and 60% respectively of the tension capacity.

4 FINITE ELEMENT ANALYSIS

4.1 Tension pile in 2D axisymmetric analysis

PLAXIS 2D axisymmetric model is used to simulate the tension pile load test, as shown in Figure 5. Weakly cemented rock and pile-soil interface are represented by the HS model with parameters provided in Table 1. The Interface elements are used to model a thin zone of intense shearing along the pile shaft between the concrete and the surrounding soil. The interface element can model slipping and gapping/overlapping behaviours, i.e., relative displacement parallel and perpendicular to the interface, respectively. The concrete pile is modelled using volume elements with a linear elastic material representing the concrete properties. The tension load is applied in increments (to simulate pile load tests) as a uniformly distributed load on the pile head. The pile-soil interface is configured by assigning a weakly cemented material with manually reduced strength and stiffness based on an interface factor of 0.67. The interface is extended beyond the pile toe to avoid stress oscillation at pile corners.

The HS model is preferred over the Mohr-Coulomb model as dilatancy cut-off is available in HS model by specifying a maximum void ratio (e_{max}). The input e_{max} is the void ratio at critical state. When e_{max} is reached, the dilation angle is set to zero (PLAXIS, 2021). A halved Young's modulus of 15 GPa is adopted for the concrete to account for reduction in stiffness due to cracks under tension loading. This would overpredict pile displacement under small load increments, but would better predict pile displacement under higher load increments.

4.2 Calibration of soil parameters

The dependency of pile axial response on the soil strength and stiffness properties, such as Young's modulus, friction angle, cohesion, dilatancy angle of the weak rock was investigated by undertaking a large number of parametric analyses. The parametric results are calibrated with the monotonic load-displacement responses of the 240 mm and 340 mm test piles. The 450 mm pile is not used in the calibration due to the difference in ground conditions from the two other piles.

Table 1: Optimal range of HS model parameters

Parameters	Weak rock	Pile-soil interface
γ_{unsat} (kN/m^3)	16	16
γ_{sat} (kN/m^3)	16	16
E_{50}^{ref} (MPa)	180 - 210	95 - 115
E_{oed}^{ref} (MPa)	180 - 210	95 - 115
E_{ur}^{ref} (MPa)	540 - 630	290 - 345
c_{ref} (kPa)	90 - 100	60 - 67
ϕ (°)	42 - 44	31 - 33
ψ (°)	16 - 22	16 - 22
e_0	0.5	0.5
e_{max}	0.53 - 0.56	0.53 - 0.56

The optimal ranges of weak rock and the pile-soil interface properties obtained in the parametric analyses are shown in Table 1. The dilatancy cut-off limited by e_{max} was found to have a significant influence on the predicted tension capacity of the pile, as shown in Figure 6. In the final analyses, an e_{max} of 0.55 was adopted.

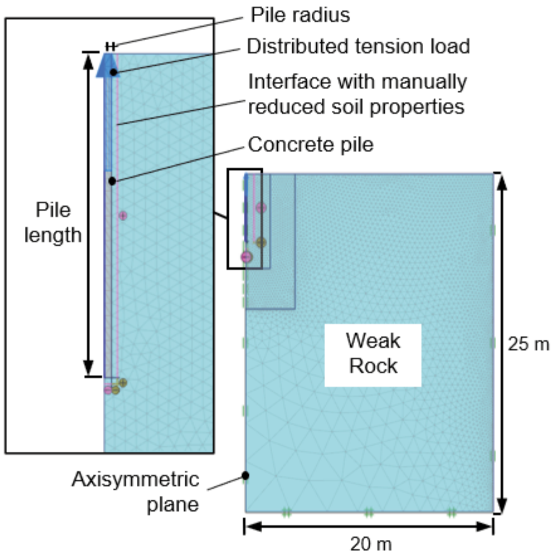

Figure 5. Tension pile load test in PLAXIS 2D

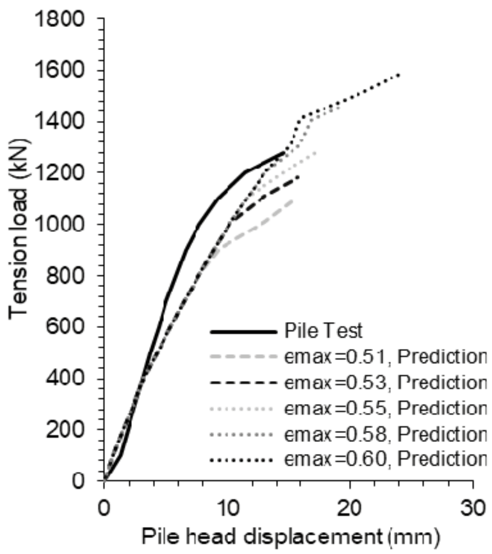

Figure 6. Parametric study of 240mm pile on maximum void ratio

5 COMPARISON BETWEEN NUMERICAL AND FIELD TEST RESULTS

5.1 Shear stress-displacement

Figure 7 compares the average shaft shear stress (τ_{avg}) versus normalised pile head displacement (w/D) between the test piles and the predictions for the 240 mm, 340 mm and 450 mm piles. The predictions for the 240 mm and 340 mm piles are in a reasonable agreement with the field test results, despite generally predicting a softer response. The predicted maximum τ_{avg} for the 450 mm pile

(300 kPa) is about twice the measured value at failure (130 kPa), and this is likely attributed to the presence of weaker material with no cementation in the field.

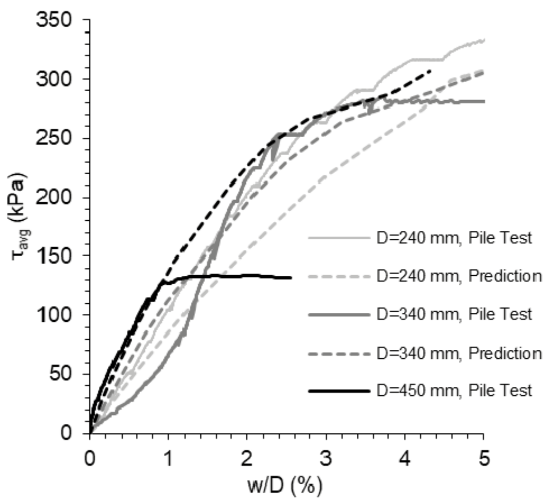

Figure 7. Comparison of τ_{avg} between measurements and predictions

5.2 Axial load, axial strain and shear stress distribution

Figure 8 compares the axial tensile load, axial strain and unit shaft friction for the 240 mm pile between the measurements and predictions. In Figure 8(a), the predicted axial load distribution is comparable with the measured distribution at smaller loads. When the tensile load at pile head is greater than 700 kN, the analysis underpredicts the load distribution between 1m and 2m depth. The results match well at the depths below 2m.

In Figure 8(b), the rate of growth in axial strain is more evident at higher loads in the measured test pile data. This is possibly due to the cracking of concrete occurring at higher loads in the field. The predictions show a relatively constant rate of growth in axial strain due to the adoption of a linear elastic material for concrete. The axial strains are overpredicted at smaller loads and better predicted at higher loads because a halved concrete modulus of 15 GPa was used throughout all load increments.

In Figure 8(c), the predicted trend of distribution of unit shaft friction is generally similar to that measured, despite underpredicting the maximum unit shaft friction values. This could be caused by the simplified ground profile modelled in the analysis, which does not capture the spatial variation of q_c observed in the field that would influence the shaft mobilization.

6 COMPARISON OF PEAK FRICTION BETWEEN NUMERICAL, CASE HISTORY AND FIELD TEST RESULTS

The peak shear stresses (τ_p) predicted from the numerical model are normalised with q_c and plotted against the q_c normalized by atmospheric pressure (p_a) as presented in Figure 9. The measured τ_p/q_c ratios from the test piles at Pinjar and their best-fit mean trend line, the suggested design lower-bound line by Riyat and Lehane (2018), and the field measurements by Joer and Randolph (1994) on grouted driven piles are also presented for comparison.

The predicted τ_p/q_c ratios are in good agreement with the best-fit mean trend line of the three test piles. Comparing to other case histories, the prediction also captures the trend of decreasing τ_p/q_c with increasing q_c/p_a.

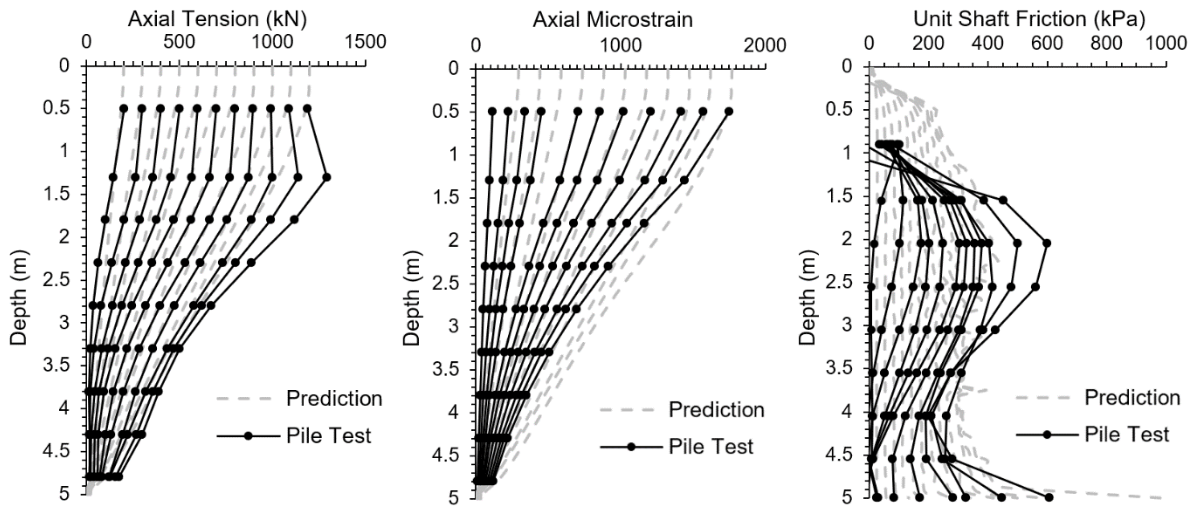

Figure 8. Comparison of axial load (a), axial strain (b) and unit shaft friction (c) in 240mm pile

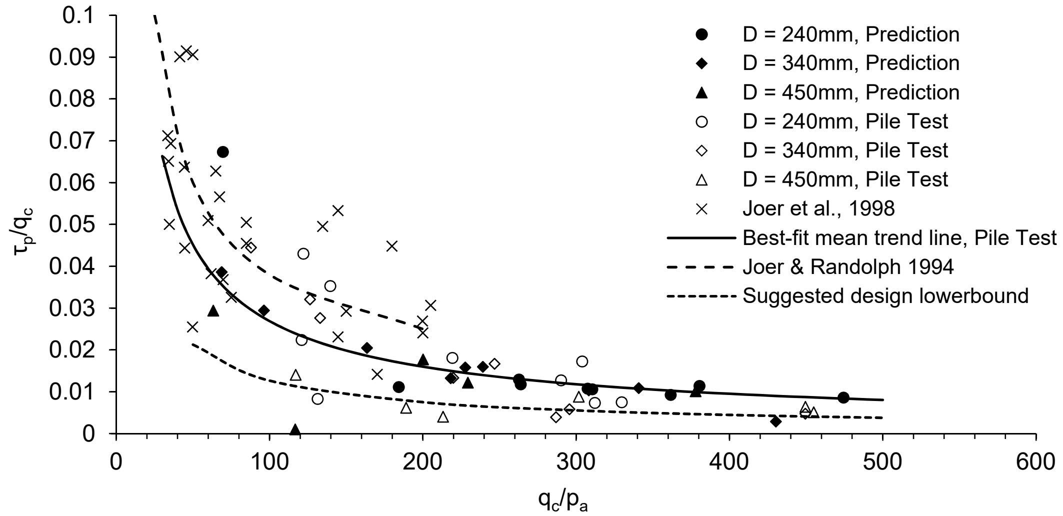

Figure 9. Comparison of τ_p/q_c ratios between test piles, numerical predictions, and case history results

7 CONCLUSION

This paper shows that the CPT results revealed a spatial variation of cone resistance q_c in the weak rock formation and the poor recovery of core samples poses the challenges in obtaining undisturbed samples for strength testing. Thus, making the dependency of design on shaft friction and CPT q_c more favourable.

This paper also presents FE 2D axisymmetric analyses in modelling three instrumented tension pile load tests at Pinjar. Parametric analyses were carried out to obtain a reliable range of soil and soil-pile interface input parameters, and calibrated against the results of high-quality soil/rock testing and pile testing. It is revealed that the dilation at the soil-pile interface affecting normal effective stress acting on the pile shaft has a significant influence on the predicted pile tension capacity. The predicted τ_p/q_c vs q_c/p_a ratios are in good agreement with the case histories in similar ground conditions and pile types.

The low pile tension capacity measured at the 450 mm test pile could be caused by a combination of a reduction in dilation with increased pile diameter and the presence of weakly cemented material.

The FE modelling could be further improved by considering the pile installation effect, pile roughness, constant normal stiffness caused by dilation at soil-pile interface, variation in ground profile, etc.

8 ACKNOWLEDGEMENTS

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Initiation of Internal Erosion in Earth Dams: A Particle-Scale Computational Approach

J. Qi[1], N. Yousefpour[2], PhD, PE, G. Narsilio[3], PhD, and M. Pouragha[4], PhD

[1]PhD Student, The University of Melbourne, Department of Infrastructure Engineering, Parkville, VIC; email: jie.qi@student.unimelb.edu.au
[2]Doreen Thomas Fellow, The University of Melbourne, Department of Infrastructure Engineering, Parkville, VIC; email: negin.yousefpour@unimelb.edu.au
[3]Professor, The University of Melbourne, Department of Infrastructure Engineering, Parkville, VIC; email: narsilio@unimelb.edu.au
[4]Assistant Professor, Carleton University, Department of Civil and Environmental Engineering, Ottawa, CA; email: MehdiPouragha@cunet.carleton.ca

ABSTRACT

Australia is known as the driest populated continent in the world, but with periods of high rainfall and flooding followed by long droughts. Earth dams are the number one supplier of water for irrigation, hydropower, and clean water, as well as the major infrastructures for flood control, amongst other purposes. Internal erosion accounts for about 50 percent of dam failures in Australia and across the world. Such failures could be catastrophic, as they often occur without noticeable precursors, posing significant risks to public safety and downstream infrastructures. In this study, we incorporate the Discrete Element Method (DEM) coupled with Computational Fluid Dynamics (CFD) to simulate soil samples under internal erosion as representative elements for dams. The outputs of simulations are evaluated using a statistical machine learning (ML) method to better assess the triggers of internal erosion based on spatiotemporal patterns in particle-scale and sample-scale parameters, such as particle velocity, particle-particle contact force, and fluid-particle coupling force, as well as kinetic and total energy during the initiation of erosion process. Understanding these patterns and correlations at the particle scale may assist in (macro-scale) engineering monitoring and mitigation strategies.

Keywords: DEM-CFD, AI-ML, Internal Erosion, Early Detection, Initiation, Earth Dams

1 INTRODUCTION

Internal erosion is a typical issue faced by water-retaining structures such as earth dams, posing a significant risk to public safety and the economy. This phenomenon frequently occurs in gap-graded soils subjected to certain hydraulic and mechanical conditions. In this context, the internal erosion in structures such as earth dams has been studied through different methods, such as field monitoring (Chen et al. 2018; Cai et al. 2020), laboratory tests, including triaxial sample tests (Richards et al. 2012; Fonseca et al. 2014) and model tests (Planès et al. 2016), as well as numerical simulations (Wang et al. 2017; Zou et al. 2020). The initiation and development of internal erosion are mainly influenced by three factors: (a) particle gradation, (b) hydraulic condition, and (c) mechanical state (Brown et al. 2009). Internal erosion usually initiates from imperceptible transport of fine particles at the pore scale with no evident signs and gradually develops into catastrophic dam breaches, which makes the early detection of internal erosion an essential demand.

The development of numerical simulation and data-driven tools such as machine learning has made it possible to gain insights into the initiation of internal erosion at the particle scale. In this research, we developed a 3D coupled Computational Fluid Dynamics (CFD) - Discrete Element Method (DEM) model of gap-graded sand samples. The relative density and small strain stiffness of the model are calibrated with a virtual simple shear test combined with empirical equations. The CFD-DEM model is subjected to a hydraulic gradient that prompts internal erosion. Some particle-scale parameters (e.g., particle velocity, particle-particle contact force, particle-fluid coupling force, particle contact number) and sample-scale parameters (e.g., mass loss), are monitored. A statistical machine learning (ML) approach using the Ridge Regression method is taken to provide particle-scale insights from the data and reveal the subtle patterns indicating the initiation of internal erosion.

2 NUMERICAL MODEL

2.1 Particle gradation and calibration

A gap-graded sand sample with Fine Content (FC) of 25% is selected for the simulation (the fine content is still sand with diameters of 0.363 mm and 0.30 mm). The gradation curve of the sand sample is shown in Figure 1. The 2 groups of coarse particles with diameters of 3.43 mm and 2.36 mm make up 37.5% of the total mass, while the 2 groups of fine particles represent 12.5% of the total mass respectively (Table 1).

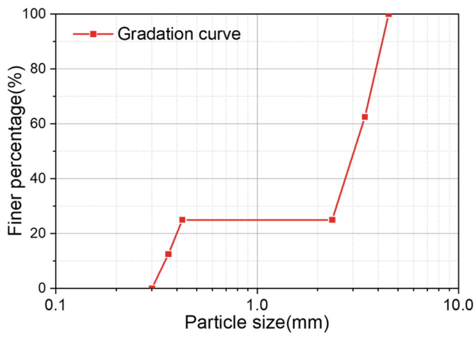


Figure 1. Particle gradation curve

Table 1. Particle size and mass fractions

Particle Group	Diameter [mm]	Relative Diameter	Mass Fraction [%]
D1	3.43	1.000	37.5%
D2	2.36	0.688	37.5%
D3	0.363	0.106	12.5%
D4	0.30	0.087	12.5%

The calibration of the DEM assembly is conducted in virtual simple shear tests. The Hertz-Mindlin model is adopted for particle-particle and particle-wall contacts. The NGI-type simple shear model with stacked rings is adopted following the numerical model of Asadzadeh et al. (2016). The assemblies are generated in a cylindrical chamber confined by 10 rigid rings with a diameter of 70 mm and a height of 20 mm. A vertical load FN is applied to the top plate of the model to compact the sample. During the simple shearing test, a horizontal velocity of 0.1 mm/s is assigned to the bottom plate while that of the top plate is fixed to 0, such that the horizontal velocity of the rings increases from 0 on the top to 0.1 mm/s on the bottom.

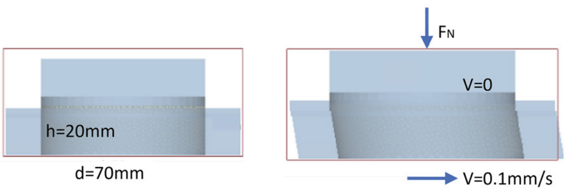


Figure 2. Simple shear test calibration model

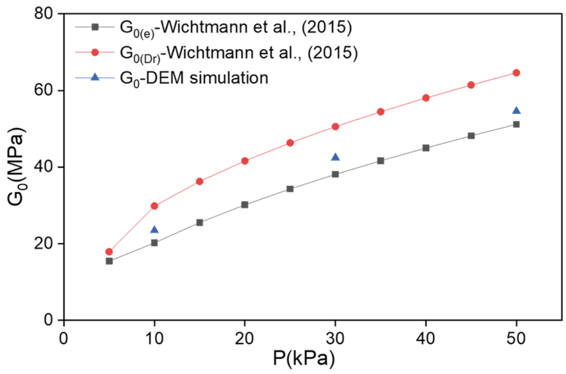


Figure 3. Prediction of G_0 with empirical equations

Table 2. Calibration model parameters

Particle properties		Particle-particle contact		Particle-wall contact	
$\rho_{partcile}$	2650 kg/m^3	μ	0.75	μ	0
E	1e8Pa	μ_r	0.3	μ_r	0
v	0.25	C_{res}	0.2	C_{res}	0.2
D_r	0.51	n	0.33		

Particle density $\rho_{particle}$; Young Modulus E; Poisson's ratio v; Coefficient of static friction μ: the ratio of static friction force to normal contact force; Coefficient of rolling friction μ_r: the ratio of rolling resistance force to normal contact force; Restitution coefficient C_{res}: ratio of relative velocity after colliding to initial relative velocity; Relative density Dr; and Porosity n.

The small strain stiffness G_0 from simple shear tests is compared with that of the empirical equations proposed by Wichtmann et al. (2015) which consider the impact of gradation and fine content (Figure 3). The $G_{0(e)}$ and $G_{0(Dr)}$ are the small strain stiffness from experiential equations based on void ratio e and relative density Dr. While obtaining G_0 from a simple shear test on anisotropic material involves errors, the measurement taken herein is to serve as a first-order approximation of the elastic stiffness. The results from the calibrated model show good agreement with the empirical formula. The calibration model parameters are summarised in Table 2 with the Young's Modulus of the particles limited to 1e8 Pa to save the computational resources (Asadzadeh et al. 2016).

2.2 CFD-DEM model

The CFD-DEM model of the gap-graded sand is created in a cylinder wall (D=15mm, H=25 mm as in Figure 4) with the same gradation (Figure 1), mass fraction (Table 1), and particle parameters (Table 2) as the calibration model. The time step of DEM is set to be 5e-7 s for the stability of calculation. For the first step, the particles are randomly generated at the top of the cylinder with predefined mass fractions (Table 1) and then settled to the bottom under gravity. After the generated particles settle down, the density of the loose sample is measured as the minimum density. To calculate the relative density, the static and rolling friction coefficients of the particle are set to 0 to achieve the dense state with maximum density. Based on the estimated minimum and maximum densities, the required density of the assembly is then calculated based on the target relative density of the sample Dr. The generated dry samples are then compacted by a plate from the top to achieve the calculated target density.

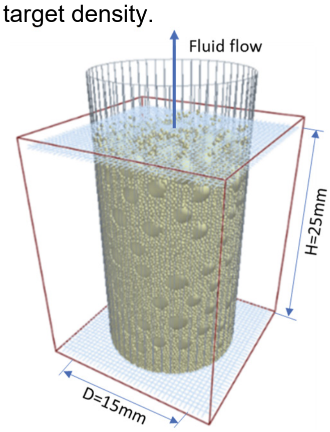


Figure 4. Dimensions of gap-graded DEM sample

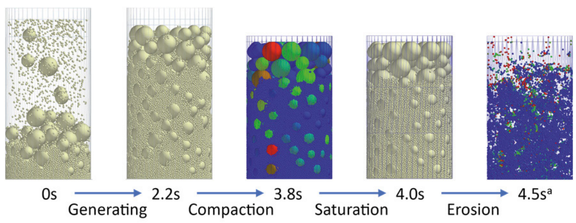


Figure 5. Sample generating and testing
[a] Coarse particles are hidden at t=4.5 s for better visualization

The Finite Volume Method (FVM) is utilized to model the fluid field. The fluid cylindrical domain has the same radius as the DEM assembly, with a larger height to enclose the top and bottom of the DEM sample. Inlet and outlet boundary conditions are assigned to the bottom and top of the fluid field respectively, while the cylinder surface is set as the wall boundary.

The particle-fluid coupling is achieved through solving locally averaged Navier-Stokes (N-S) Equations with the volume fraction ε of the liquid phase calculated in each cell as shown below (Li et al. 2012):

$$\frac{\partial(\varepsilon\rho)}{\partial t}+\nabla\cdot(\varepsilon\rho\mathbf{u})=0 \tag{1}$$

$$\frac{\partial(\varepsilon\rho\mathbf{u})}{\partial t}+\nabla\cdot(\varepsilon\rho\mathbf{u}\cdot\mathbf{u})=-\nabla p+\nabla\cdot(\eta\varepsilon\nabla\mathbf{u})-\varepsilon\rho\mathbf{g}-\mathbf{S} \tag{2}$$

where ε is the volume fraction of fluid; ρ is the fluid density, ρ=1000 kg/m^3; t is time; **u** is the fluid velocity vector; η is the fluid viscosity, η**=**0.001003 Pa·s, p is the pressure; **g** is the gravity vector, and **S** is the momentum sink.

Force and momentum exchange between the solid and fluid phases is then calculated. The fluid cells are set to be coarser than the largest particle size to avoid 0 values for ε which can cause errors in locally averaged N-S equations. The time step for fluid calculation and information exchange between two phases is 5e-5 s for calculation stability.

The compacted dry sample is then coupled with CFD under static hydraulic pressure for saturation. Both the DEM and CFD simulations continue to reach equilibrium during the saturation process while keeping Dr constant. For the erosion stage, the plate on the top is replaced by a mesh that is only permeable to the fine particles. The hydraulic pressure of the outlet is set to 0, while that of the inlet is fixed to a value that gives a hydraulic gradient of 1 on the whole sample. The fine particles are then eroded from the pore space of coarse particles driven by the hydraulic gradient. The eroded particles are deleted when they are beyond the red rectangular domain of DEM (Figure 5).

3 RESULTS

3.1 Transport of fine particles

After the application of hydraulic gradient at t=4.0 s, the fluid-particle coupling force increases from the values of buoyancy to a higher and steady value within 0.1s for both D3 and D4 (fine) particles (Figure 6(a)). As expected, the loss of fine particles for the D3 group occurs later than that of D4 (Figure 6(b)), due to its relatively larger mass. The progress of internal erosion is calculated as the ratio between the number of eroded particles and the total number (Figure 6(c)). The progress of internal erosion based on lost particles happened after 4.1 s for both groups. However, the erosion in D4 group (fine) particles starts earlier and progresses faster than that of D3. This is also caused by the difference in particle size and mass between the two groups.

The transport of D4 fine particles in the sample is shown in Figure 7 with the particles coloured by velocity-z(a) and fluid-particle coupling force(b). The velocity distribution of particles at 4.5 s indicates the transport of fine particles with an increased rate at the top of the sample. The values of coupling force are dispersed to some extent as the porosity calculated in different fluid cells varies. This mechanism may be the reason the transport of neighbouring fine particles shows different patterns.

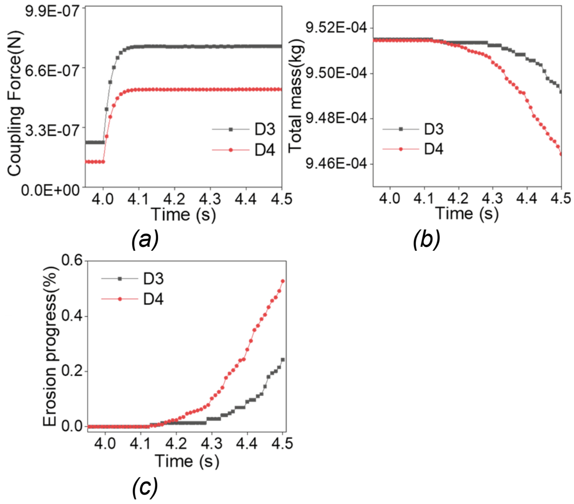


Figure 6. Evolution of averaged particle parameters (a) fluid-particle coupling force of D3 and D4; (b) particle total mass of D3 and D4 (c) erosion progress of D3 and D4

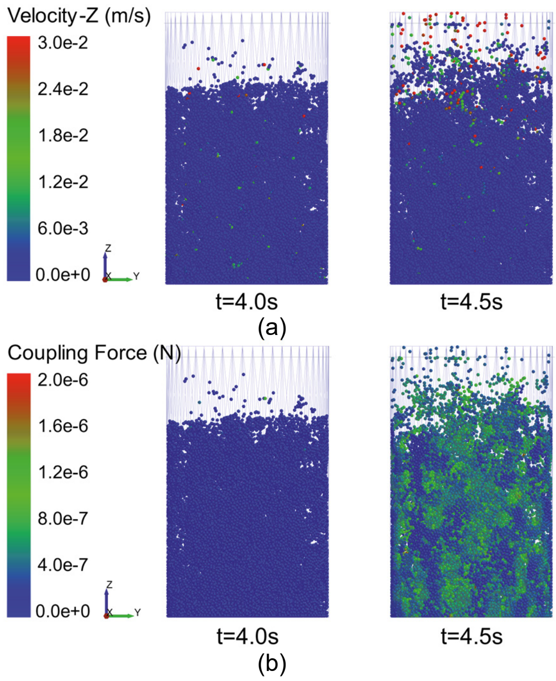
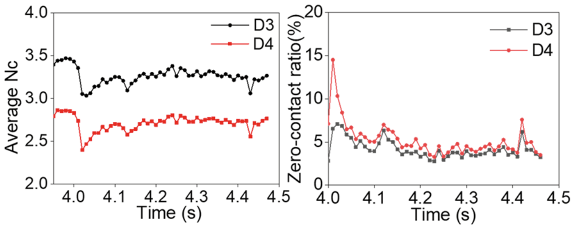


Figure 7. Transport of D4 fine particles (a) particle velocity-z distribution; (b) fluid-particle interaction force distribution

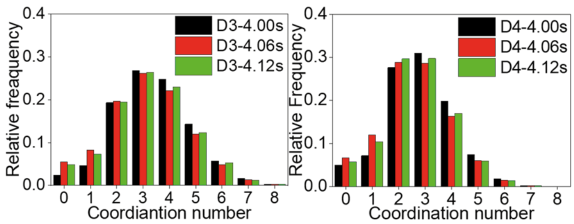
3.2 Evolution of particle-scale parameters

The coordination number quantifies the number of contacts between the target particle and the adjacent ones. After the application of hydraulic gradient, the average coordination number sharply dropped for both D3 and D4 fine particles (Figure 8(a)). The average contact numbers gradually converged to a steady value, which is consistent with the observation of Kawano et al. (2017). The zero-contact ratio of the fine particles is calculated from the ratio of the particles with zero contact numbers over the total number of particles remained within the sample in that group (i.e., total number of particles - number of eroded particles). Both the zero-contact ratio of the D3 and D4 group of fine particles sharply increased after t=4.0 s and then decreased to a relatively low value. The peak of D4 is higher than that of D3, as expected, as the finer particles are subjected to a higher rate of erosion.



(a) (b)

Figure 8. Evolution of particle contact state (a) average coordination number of D3 and D4; (b) zero contact ratio of D3 and D4



(a) (b)

Figure 9. Statistical distribution of particle coordination number (a) D3 group fine particles (b) D4 group fine particles

The statistical distribution of particle coordination numbers for the D3 and D4 group fine particles at t=4.00 s, t=4.06 s, and t=4.12 s are shown in Figure 9. The relative frequency of low coordination numbers at both t=4.06 s and t=4.12 s increased compared with that of t=4.00 s. This indicates more particles lost contact with adjacent particles and started to be eroded from the sample. The decrease of coordination number can be a parameter indicating the initiation of internal erosion at the particle scale.

3.3 Assessment of particle-scale parameters on internal erosion

The particle-scale parameters could reveal the initiation of internal erosion. However, due to the large number of particles and the associated parameters, plus their temporal variations and interactions among them, it is challenging to understand the embedded internal erosion patterns and make interpretations directly from the data. Therefore, we have applied statistical machine learning to analyse the particle-scale behaviours associated with the initiation of internal erosion. For this analysis, the particle-scale parameters of the D4 group are incorporated throughout the erosion process (from 4.0 s to 4.5 s) with a time resolution of 0.01 s, including 13 parameters associated with 23,200 fine particles incorporated (Table 3).

The scikit-learning library (Pedregosa et al. 2011) is utilized to calculate the importance of the particle-scale parameters through Ridge regression approach. The decrease of coordination number at each time step is used as a measure of internal erosion and is defined as the response (target) variable in equation (3). The 13 particle-scale parameters for all the D4 particles are considered as the predictor variables forming the 13 x 23200 matrix in equation (4). The ridge regression is then used to fit the multiple-linear regression model between the predictors, $X_{(t,k,n)}$ and the response variable, $y_{(t,n)}$ at each time step, t throughout the simulation. All the variables are normalized before regression:

$$y_{(t,\,n)}=N_{\mathbf{c}(t_0=4.0,n)}-N_{\mathbf{c}(t,\,n)} \tag{3}$$

$$X_{(t,k,n)}=\begin{bmatrix} X_{(t,k=1,n=1)} & \cdots & X_{(t,k=13,n=1)} \\ \vdots & \ddots & \vdots \\ X_{(t,k=1,n=23200)} & \cdots & X_{(t,k=13,n=23200)} \end{bmatrix} \tag{4}$$

$$y_{(t,n)}=X_{(t,k,n)}\cdot \mathbf{B}_{(t,k)}+e_{(t,k)} \tag{5}$$

where t∈{4.01, 4.02⋯4.50 s}; k∈{1, 2⋯13} is the ID of particle-scale parameters; n∈{1, 2⋯23200} is the fine particle ID number; $N_{\mathbf{c}(t,n)}$ is the coordination number of particle number n at time step t; $y_{(t,n)}$is the decrease of coordination number for the n'th particle at time step t; $X_{(t,k,n)}$ is the k'th parameter of the n'th particle at time step t; $\mathbf{B}_{(t,k)}$ and $e_{(t,k)}$ are regression coefficients and errors of the ridge regression.

The regression coefficients are then evaluated to compare the contribution of each particle-scale parameter to the decrease of coordination numbers. The importance coefficients of the parameters are ranked based on averaged regression coefficients throughout the time steps (Figure 10). The particle velocity magnitude (X7) and kinetic energy (X8) are the most important parameters influencing the reduction of coordination numbers (parameters that are directly related to the transport of fine particles). The particle-particle contact force (X11) is another essential parameter, as the contact forces from adjacent particles restrict the floating of the target particle. Based on the results of the regression, the total energy (X10) is another important factor contributing to erosion. The potential energy of particles is mainly determined by their Z-position, which makes it show similar importance as the particle position on the Z-axis (X3).

The evolvement of the importance coefficient for the four most significant parameters (X7, X8, X10, X11) together with fluid-particle coupling force throughout the simulation time steps are shown in Figure 11. The fluid-particle coupling force (X12) has a non-negligible influence on the reduction of coordination numbers at the beginning; however, it shows no significant influence for the rest of the time. This is because the fluid-particle coupling force disturbed the initial mechanical balance of fine particles. The mechanical and dynamic response of fine particles then causes the drop of coordination number in Figure 8(a), when the coupling force keeps relatively stable (Figure 6a)) hence its low importance coefficient thereafter. The temporal trend of the coefficients for the top 4 important parameters shows similarities; they slightly increase at the start of erosion (4.0 s ~ 4.02 s) and sharply fluctuate after t=4.1 s. This might be caused by the interaction between fine particles driven by fluid flow and the skeleton of coarse particles.

Table 3. Particle-scale parameters used to define the D4 fine particles

ID	Parameter	Units
X1	X: Particle Position X	mm
X2	Y: Particle Position Y	mm
X3	Z: Particle Position Z	mm
X4	V_X: Particle Velocity X-component	mm/s
X5	V_Y: Particle Velocity Y-component	mm/s
X6	V_Z: Particle Velocity Z-component	mm/s
X7	V_M: Particle Velocity Magnitude	mm/s
X8	E_K: Particle Kinetic Energy	J
X9	E_P: Particle Potential Energy	J
X10	E_T: Particle Total Energy	J
X11	$F_{P\text{-}P}$: Particle-Particle Compressive Force	N
X12	$F_{F\text{-}P}$: Fluid-Particle Coupling Force	N
X13	D_Z: Z-Displacement	mm

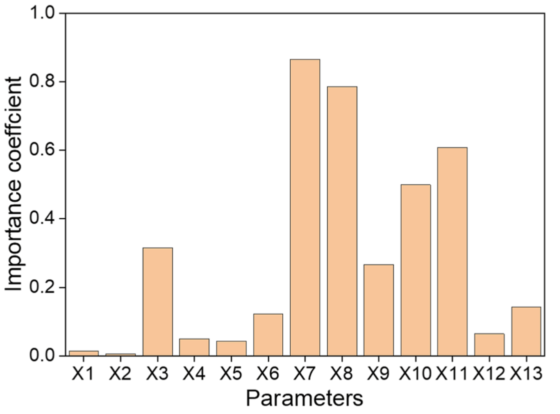


Figure 10. Parameter average importance coefficients

The average values together with confidence intervals of particle-scale parameters from D4 particles are shown in Figure 12. Both the velocity and kinetic energy sharply increase at the initial stage and gradually settle to a lower value with considerable fluctuations. This might be caused by the sudden application of the hydraulic gradient on the sample. The total energy of the particles gradually increases and approaches a relatively stable value as both the increase of particle velocity and z-position (gravitational potential energy) contribute to the total energy. The particle-particle contact force decreases throughout the time steps. The trend shown in Figure 12(a) and (b) is consistent with that in Figure 8 as both the increase-decrease stage and fluctuation stage appear at roughly the same period. The fluctuation stage in Figure 11 also shows a similar trend as in Figure 8 and Figure 12. This similarity reveals the hydraulic gradient is the main influencing factor at the very beginning, while the interaction between fine-and coarse particles contributes more to the fluctuation stage.

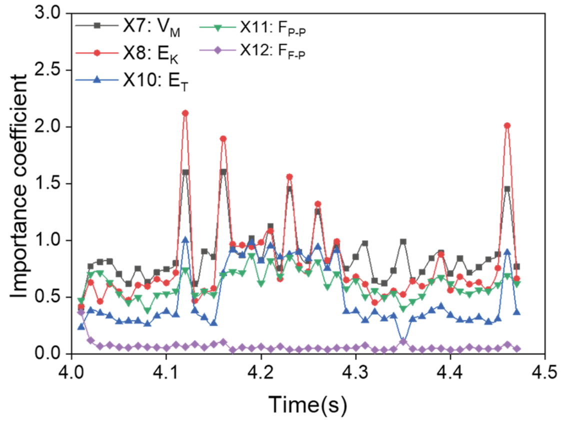


Figure 11. The evolvement of parameter importance coefficients

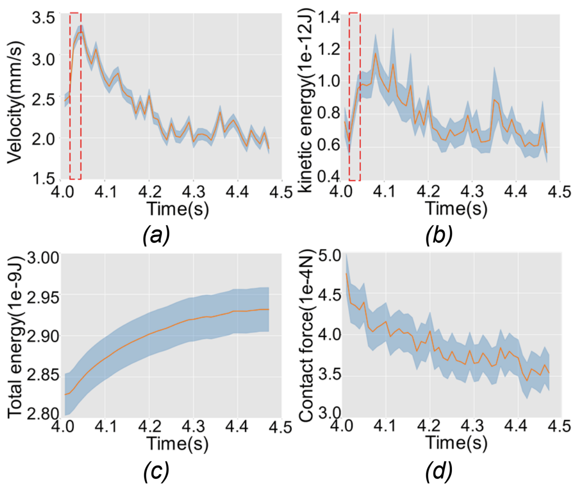


Figure 12. Development of significant parameters(a) particle velocity magnitude V_M; (b) particle kinetic energy E_K; (c) particle total energy E_T; (d) particle-particle compressive force $F_{P\text{-}P}$

3.4 Analysis of internal erosion initiation

The transport of fine particles happened throughout the sample, but higher velocities were observed at top of the sample where the mesh sits. The average coordination numbers of both D3 and D4 group particles deceased shortly after the application of the hydraulic gradient and gradually recovered to a steady value and the relative frequency at low coordination numbers increased after the hydraulic gradient was applied, showing the internal erosion has started. However, the mass loss did not occur at the initiation of the internal erosion as only when the eroded particles move out of the sample boundary the change of mass could be observed. This indicates that mass loss cannot be used to detect the initiation of internal erosion.

Based on the observation of particle-scale and sample-scale parameters, the internal erosion started shortly after the application of the hydraulic gradient at around t=4.02 s to t=4.0 5 s (4e5~1e6 time steps after applying HG). The considerable decrease of average particle coordination number and the increase of particle velocity both reveal that fine particles are losing contact with other particles and are escaping from the assembly around this time. The gradual recovery of the coordination number shows the disturbance of fine particles caused by the hydraulic gradient fades after t=4.02 s. The fluctuations of the average coordination number after t=4.10 s are mainly due to the mechanical interactions between fine particles and coarse skeletons. This mechanism also caused the fluctuations of particle velocity, particle kinetic energy, and particle-particle contact force after the initiation of internal erosion.

4 CONCLUSION

This research investigates the initiation of internal erosion based on CFD-DEM simulations with a machine learning application. A gap-graded particle assembly is generated with 4 groups of particles of different sizes. A locally averaged CFD-DEM coupling method is utilized to simulate the fluid-particle interactions. Both the sample-scale and particle-scale physical parameters are evaluated to capture the subtle patterns associated with internal erosion.

The mass loss cannot be used to detect the initiation of internal erosion as the erosion starts earlier than the time particles are actually eroded from the sample. This is why the subtle patterns of the particle-scale parameters are more relevant for the assessment of the internal erosion initiation.

The decrease in the coordination number of fine particles is defined as the response variable associated with the internal erosion initiation. Based on the Ridge regression analysis, the particle velocity magnitude, kinetic energy, particle-particle contact force, and particle total energy were found to be the most important particle-scale parameters influencing internal erosion. The fluid-particle coupling force has also shown a non-negligible (but not significant) influence on the initiation of internal erosion. Based on the observation of particle-scale and sample-scale parameters, the internal erosion started shortly after the application of the hydraulic gradient at around t=4.02 s to t=4.05 s. However, the progression of internal erosion based on the loss of particles started after 4.1 s.

Further in-depth analyses using deep learning methods will be performed in the upcoming stages of this research to better identify pattern changes within the particles' mechanistic behaviour at the initiation of erosion to derive more relevant criteria for real-scale evaluation of internal erosion initiation in earth dams.

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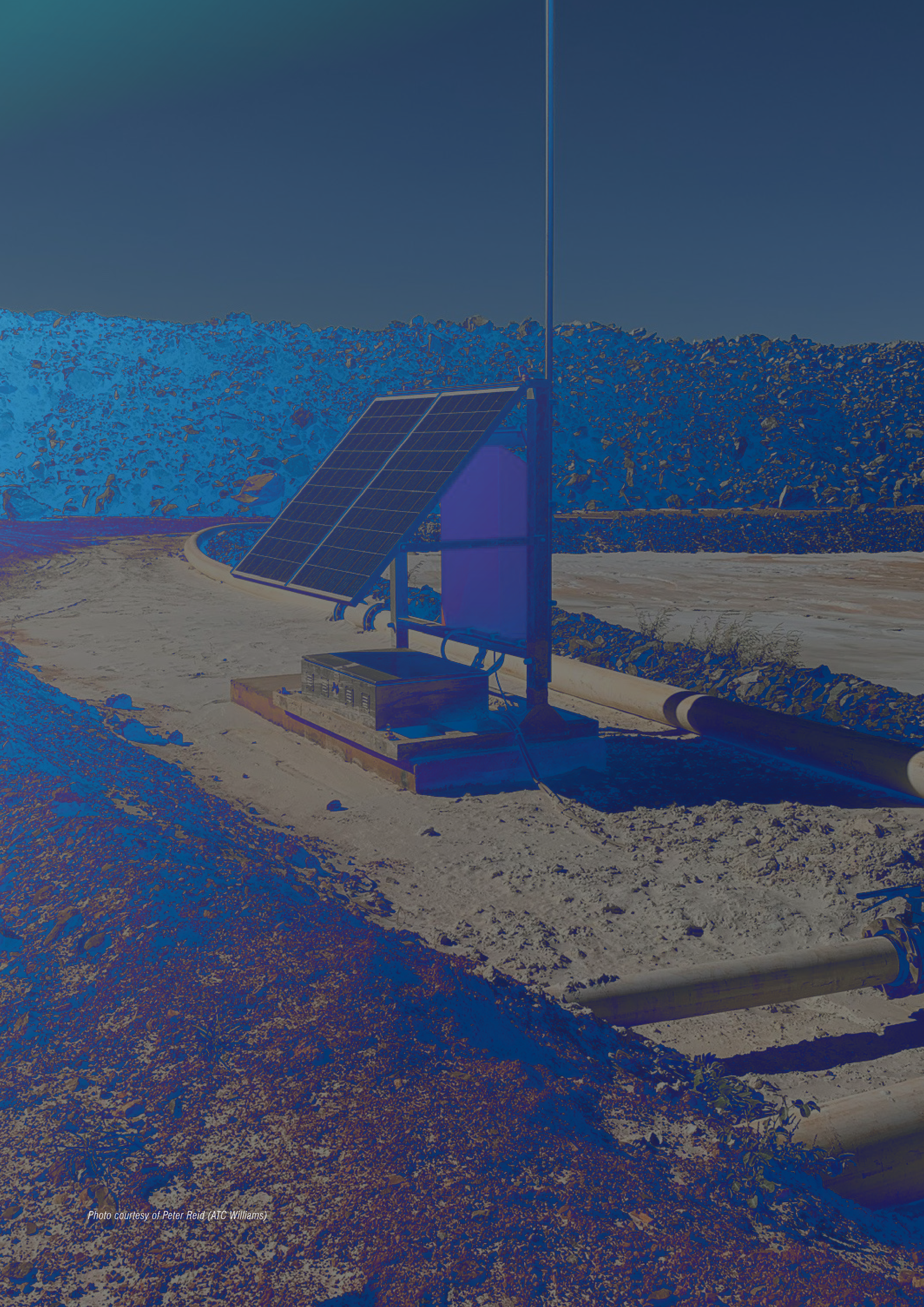
SESSION 4

DIGITAL APPLICATION AND CHALLENGES



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Challenges to Digital Transformation in Geotechnical Engineering

Q. Hayat[1], J. W. Muir[2] and H. E Nelson[3]

[1]Principal, Aurecon, 8/850 Collins St, Docklands VIC 3008; email: qaiser.hayat@aurecongroup.com
[2]Ground and Underground Engineering Practice Leader, Aurecon, 8/850 Collins St, Docklands VIC 3008; email: jack.muir@aurecongroup.com
[3]Associate Engineer, Aurecon, 8/850 Collins St, Docklands VIC 3008; email: hamish.nelson@aurecongroup.com

ABSTRACT

The application of digital data and technologies in geotechnical engineering is not new; however, the use is sporadic and inconsistent among geotechnical engineering specialists and firms across the world. Further, this use is often limited to specific areas such as processing of field data during ground investigations and detailed numerical analyses of complicated foundation systems during design. There is a general lack of interoperability among the various digital tools and systems, which is inhibiting efficiency that could have been achieved otherwise using compliant platforms which can readily transfer data and models. This paper examines by way of examples where digital data and technologies are being used to increase efficiency in design and construction and enhance collaboration on construction projects. The paper explores the current state of the art and future potential opportunity to automate the whole geotechnical design process of capturing field data, generation of ground models, analyses and design, and visualisation of final solutions without the need for manual data entry at any of the intermediate stages. The level and stages where human input would continue to be required in geotechnical engineering are also reviewed, given the level of empiricism we still adopt due to significant gaps in our understanding of the variability of natural materials, behaviour of ground under loading and due to its complex interaction with other man-made materials and structures. In addition, a view is taken of the market driven barriers that might be limiting the efficiency of the geotechnical industry and the society in general, which could gain from the ongoing digital transformation.

Keywords: geotechnical, digital, market, challenges, design, construction, risk

1 INTRODUCTION

Digital transformation in geotechnical engineering broadly engenders a feeling of hope in the geotechnical engineers due to its potential strengths. Digital tools and processes can be used by geotechnical engineers to capture the spatial distribution and highly complex characteristics of natural materials; to facilitate prediction of the mechanical behaviour of materials for design purposes and to help visualise a ground model and associated geotechnical solution that can otherwise be difficult to communicate to the end user and non-geotechnical engineers. However, on the other hand, efforts to inculcate the widespread use of digital technology in geotechnical engineering process have yet to meet noteworthy success due to several impediments and challenges. These challenges are both of commercial and technical nature. The main commercial challenges include lack of competition among digital solution providers and geotechnical community itself (supply) as well as lack of appreciation among the end users or clients (demand) of the value that digital transformation can bring to their projects. Impediments of technical nature include uncertainties of dealing with significant variation in characteristics of natural materials (soil and rock), as well as limitations of our understanding of their highly complex constitutive behaviour when compared to other fields of engineering which mainly deal with manmade materials that can be modified to a required specification for design purposes. Due to these factors, it is relatively difficult to reduce the required human intervention and judgement in geotechnical design processes and to rapidly bring digital transformation in geotechnical engineering so that it is on a par with other engineering fields.

In this paper, the current state of digital transformation is reviewed, some of the known impediments are discussed and recommendations are provided to deal with the challenges.

2 STATE OF THE ART

2.1 Data Capture and Storage

Geotechnical Engineering Industry has achieved some good and consistent progress in data capture and storage processes over the last two decades or so, specifically when it comes to the transfer and storage of digital geological information from ground investigations.

Perhaps the most widespread use of digital data transfer within the international geotechnical community has been the AGS data transfer format, which was setup by Association of Geotechnical & Geoenvironmental Specialists in 1991. The AGS Data Format, with periodic updates allows for a consistent and seamless sharing of factual data between the different software used within the geotechnical industry (https://www.ags.org.uk). Recently, AGS have introduced AGSi (https://www.ags.org.uk/data-format/agsi-ground-model/) for ground models and interpreted data. Further, AGS piling (http://www.ags.org.uk/data-format/ags-piling-draft/) has been initiated to capture piling data.

GeoSciML is another example in the data space, which is an international data transfer standard for geological data, developed by the IUGS Commission for the Management and Application of Geoscience Information (CGI). The Government Geoscience Information Committee (GGIC), which represents all the Australian and New Zealand government geological surveys, has endorsed GeoSciML as the Australasian geological data transfer standard (https://www.ga.gov.au/).

For site investigations desktop studies, geological maps are digitally available to public in most states in Australia. Australian Government's Geoscience Australia Portal (https://portal.ga.gov.au/) provides full access to Geoscience Australia data and other publicly available geological data sources. It also provides a suite of analytical and multi-criteria assessment tools to maximise the value of the data. Berg et al. (2011) suggest Geoscience Australia and Geoscience Victoria have been the most active Geological Survey Organisations in Australia.

With the increasing availability of affordable Unmanned Aerial Vehicles (UAVs) / drones becoming available to "access" places otherwise not easily accessible or inherently unsafe on foot. In many cases, they can be used to undertake inspections of cliff faces without the need for scaffolding or roped access. The key limitation being that one can only see the rock without being able to touch or feel it, but often, some level of ground truthing through limited walkover surveys can significantly reduce this deficiency. Photogrammetry techniques enable accurate 3D digital models to be developed from the photographs taken from UAVs.

2.2 Development of Ground Models

Three dimensional geological models are increasingly being used to characterise ground supporting complex structures. Soil, rock, geological structures, such as faults, rock fall zones and slips are often best examined in a 3D domain.

Several geological modelling tools are available in the market with popular software used on Civil Engineering and infrastructure projects, including Leapfrog Geo, gINT and holeBASE. More specialised geological modelling tools are in use by the mining and energy sectors such as IRAP RMS suite and GoCAD Mining Suite. Similarly, specialised hydrogeological modelling tools such as Feflow and Modflow are also available. Berg et al. (2011) provides a useful global synopsis of the available geological modelling software and the list keeps on extending since then. GIS software such as Mapinfo and ArcGIS are also regularly used for 2D and 3D visualisation of site geology.

Table 1: Typical examples of Geological Modelling Software

Usage	Digital Software	Provider
Ground Model	gInt	Bentley Systems
Ground Model	HoleBASE	Bentley Systems
3D Ground Model	Leapfrog Geo	Bentley Systems

2.3 Geotechnical Analysis and Design

In the geotechnical design phase of a project, not much has changed in terms of the use of some main digital software tools for analysis and design over the past 20 years or so. The use is restricted mainly to few famous analysis software such as those required for 2D and 3D limit equilibrium, and 2D and 3D finite element analysis. Software packages are available to resolve geotechnical problems such as slope stability, ground settlement, pile group analysis, gravity and embedded retaining walls, deep excavations, driveability assessment and liquefaction analysis. Some new analysis and design software tools have been introduced into the market, but there remains just a handful of developers or providers in the geotechnical market whose software is being used by majority of the commercial firms.

Some good progress has been made to make the software tools more user-friendly and increased computing power has allowed improvement in computing speed and graphical user interface. However, very limited progress has been achieved in making the available software more interoperable.

Table 2: Typical examples of Geotechnical Design Software

Usage	Software	Provider
Slope Stability	Slope/W	Bentley Systems
Various (Finite Element)	Plaxis	Bentley Systems
Retaining Walls	Wallap	Bentley Systems
Settlement Analysis	Settle 3	RocScience
Slope Stability	Slide	RocScience
Rockfall Analysis	RocFall	RocScience
Pile Axial/Lateral	RSPile	RocScience
Pile Group	Group	Ensoft
Axial Pile Capacity	A-Pile	Ensoft
Lateral Pile Capacity	L-Pile	Ensoft
Pile Group Design	Repute	Geocentrix
Retaining Wall	ReWard	Geocentrix
Liquefaction	Liquify Pro	CivilTech
Piles	AllPile	CivilTech
Pile Driveability	GRLWEAP	Pile Dynamics
Piles PDA	CAPWAP	Pile Dynamics

2.4 3D Visualisation of Solutions

Digital visualisation of geotechnical solutions in 3D domain has limited use in the industry but projects with large earthworks infrastructure are making increased use of 3D models during construction for communication purposes among the geotechnical designers, earthworks contractor and the client.

For cut slope design, especially in rocks which require mapping by geologists, a 3D Leapfrog model with Esri's ArcView GIS interface using drone technology has been used successfully during the design and construction of the Manawatū Tararua Highway in

New Zealand. A 3D digital model for the entire project has been developed by Aurecon, including a fly-through during the tender stage which assisted the New Zealand Transport Agency and the local Iwi community leadership to understand the designer's thinking and enabled them to see first-hand how the new highway will integrate into its geological cut slope profiles and environmental context. Virtual Reality (VR) has also been an invaluable tool in conversations with both the local population of Iwi and the ultimate client, the Transport Agency. Within the Alliance, Aurecon's design and construction teams have made use of VR to help them understand how key design elements will integrate and fit in real life – for example, it has helped understand the complex interaction between bridge foundations, significant scale of earthworks, drainage and cultural expression design elements. The 3D models significantly assisted in interaction between the geotechnical engineers and other disciplines.

Figure 1. Digital Visualisation of major cuts on Te Ahu A Turanga – Manawatū Tararua Highway, New Zealand

2.5 Construction Management

Geotechnical design solutions are mostly based on idealised ground models that need to be validated on site in terms of the assumed stratigraphy, and strength and stiffness parameters. This allows the designer to provide economical designs that are not based on worst case ground conditions but the "most credible". The verification of the design requires an on-site presence as well as instrumentation and monitoring during construction. Observational method of geotechnical design is an extreme case example of this scenario, which heavily relies on site observations, and instrumentation and monitoring regime during construction.

A combination of 3D geological digital model, 2D or 3D GIS visual interface and data capture through regular fly-through using drone technology on site allows for management of construction of major earthworks on infrastructure projects. On Manawatū Tararua Highway project, monitoring of cuttings and embankments through drones alongside digital data capture tools on site have been used to feed into a 3D GIS interface in Esri's ArcView. The backend geological model in Leapfrog is then used by the office-based geotechnical engineers to compare with the face mapping results and the model is updated where needed. Verification of design assumptions and any recommendations to modify the design are all provided to the construction team in real time.

This augmented reality, where models and the real world can be visualised together by the decision makers is possible with the available digital tools but could be expanded to other projects if the wider industry adopts its use.

2.6 Instrumentation and Monitoring

Instrumentation and monitoring can assist with the design, construction, and operation of most geotechnical assets. It adds significant value when a high degree of uncertainty is present in the ground conditions, which significantly interacts with the structure being constructed. Geotechnical instrumentation can also be used for operational risk management of a potential geotechnical hazard through the use of an early warning system (EWS).

The last decade has seen the development of cloud-based systems, which can store a vast amount of data generated from monitoring of geotechnical earthworks and structural foundations during and after construction. The information can be made available to designers and to the relevant stakeholders in real time where the effects of ongoing excavations, foundation loading, retaining walls under construction and effects of weather events, for example, can be discussed and decisions made. Real-time data capture, remote monitoring with data loggers and presentation through digital visualisation platforms allow effective, timely and targeted assessment and design of remedial solutions. Spatial survey techniques, and instruments such as inclinometers, vibrating wire piezometers, extensometers, crack meters and digital profile gauge meters are commonly used on large infrastructure projects in Australia for these purposes.

2.7 Geotechnical Risk Management

Managing of geotechnical risks on projects is crucial for the success of any Civil Engineering project. Many Australian infrastructure projects have gone over budget, with a 2020 Grattan Institute report indicating cost overruns of $24 billion on just six current projects (Terrill et al., 2020). Deficiencies and shortcomings in design, ground investigations and the geotechnical interpretation of results are responsible for approximately one third of the total cost for errors (Nylén 1996 & 1999). Unforeseen site conditions are one of the most significant risk factors in causing delays to construction projects (Mbachu and Taylor, 2014).

A general perception within the construction industry seems to be that a higher budget allocation to ground investigation and testing on a project would proportionally improve the project team's understanding of ground and thus mitigate geotechnical risks to the required levels. Although true to an extent, this is just part of the story as capturing and understanding geotechnical risks is much more complex than merely capturing results from in situ and laboratory testing. As an example, based on data from UK's highway projects, around 1% of total construction cost at tender is allocated to ground investigations and even then, the data suggests that cost overruns of up to 100% might be possible

(Figure 2, Clayton, 2001). According to Jaksa (2014), expenditure on geotechnical site investigations on other types of construction projects varies considerably and can be as low as between 0.1 and 0.3% of the total project cost.

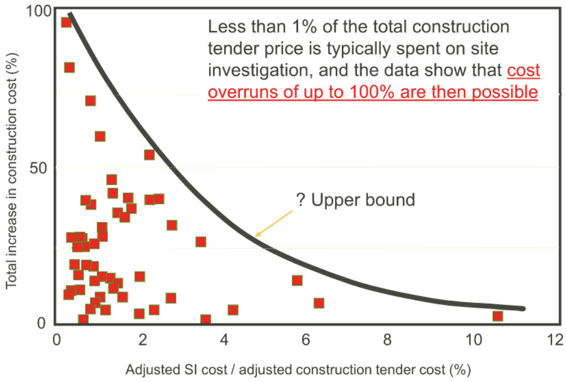

Figure 2. Adequacy of Site Investigations (Mott MacDonald and Soil Mechanics (1994), as referenced in Clayton 2001)

Geotechnical risk is a combination of mainly two types of uncertainties associated with the natural ground (Figure 3). These are risks associated with natural randomness of soils and rocks (Statistical Risks) and risks associated with inaccuracies in our prediction and estimation of reality (Systemic Risks). As such, any significant investment on site investigations beyond a certain optimum limit without properly understanding and mitigating systemic risks also, is unlikely to result in the desired positive outcome.

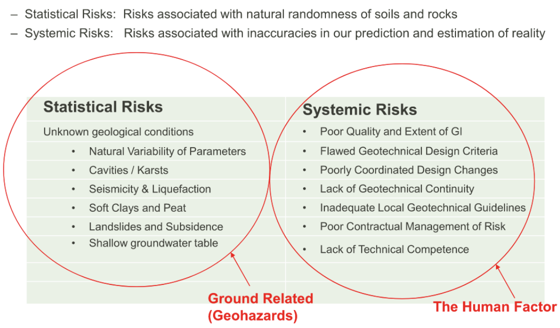

Figure 3. Types of Geotechnical Risks

The current state of the art mainly focuses on digitising information that supports understanding of statistical risks mainly and lack a holistic and systematic approach that captures both types of geotechnical risks during the course of the project.

A qualitative or a quantitative risk assessment can be carried out in practice to assess geotechnical risks; however, the general practice remains to be the assessment of geotechnical risks using a qualitative approach. This is usually done using simple Excel based matrix that determines risk levels using a product of impact and likelihood of the identified risks and communicated using a colour coded ranking system in a risk register for the purposes of mitigation analysis. Use of a more refined quantitative approach, such as Monte Carlo Simulation using digital tools, has yet to gain popularity within the geotechnical industry, although this approach is widely used by contractors on large-scale infrastructure projects.

3 CHALLENGES AND GAP ANALYSIS

3.1 Economics and Competition

Geotechnical engineering is not seen by many as an exact science, but more as an art with significant empiricism involved throughout the design process. As Professor John Burland (2012) says, “With a material as complex as the ground, empiricism is inevitable, and it is (and will always remain) an essential aspect of ground engineering”. Considering this, digitising geotechnical engineering is expected to come with a higher challenge and investment when compared to other engineering fields that have better grounds to formulate into a computer-based routine.

Digital transformation has shown to come with increased efficiency in the construction industry and a general expectation from the clients is a decrease in design time and cost on their projects. However, although geotechnical engineering industry has reaped some benefits when it comes to savings on labour intensive data entry tasks, such as those involving geological data capture, these benefits have not been transformed into an overall efficiency in the interpretation and design process and have not yet resulted in benefits to the ultimate clients in terms of shorter programme and reduced cost. This has resulted in general scepticism among the clients (low demand), and a reluctance to invest in digital transformation within the geotechnical industry.

Another impediment is the low level of competition within the geotechnical industry itself to transform the geotechnical design process due to a general lack of appetite to embrace the change. One reason for this is the lack of leadership mainly arising due to digital skill gaps within the generally more influential senior members of the geotechnical community. Most experienced geotechnical engineers who are currently leading teams within major consultant and contracting firms may not have the right level of digital skills. It may not be reasonable or even be realistic to expect every ground engineer to have a similar level of digital skills however, to speed up the transformation, it is crucial for experienced ground engineering experts to be able to communicate with the digital specialists to help progress the transformation.

3.2 Digital Twin

As Seequent (2021) defines it, “A digital twin is a digital representation of the physical system. In addition to the initial geologic model created for the site, the digital twin must also capture the evolving site conditions. A classic example would be the creation of ‘as-built’ drawings for the construction of an earthen dam or tailings storage facility”.

The scale of natural variation in ground is known to be so significant when compared to man-made materials, such as concrete and steel, that the concept of Digital

Twin in geotechnical engineering seems to be a misnomer to many in the industry. The question being asked is "Can the earth really have a Digital Twin"? As Dr. Karl Terzaghi said, "Nature has no contract with mathematics, she has even less of an obligation to laboratory test procedures and results". As an example, the variation in strength of ground (soil and rock) might vary by a factor of 50 or more compared to a general variation by a factor of 2 to 3 in concrete and steel (Figure 4). Further, additional testing data in soils does not necessarily increase accuracy of measurement of mean strength as can be seen in case of concrete (Figure 4).

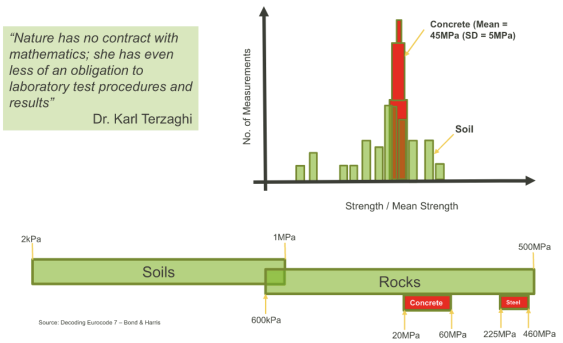

Figure 4. Natural Variability in Ground (after Bond et al., 2008)

Despite the high level of uncertainty in soils and rock, the concept of having a digital twin has a lot of promise, as long as the inherent uncertainties and limitations are well understood and catered for in the model. For example, digital twin models should capture additional data related to uncertainties associated with each individual soil or rock layers in terms of its potential variation in both vertical and horizontal domains. Similarly, each individual test parameter and the corresponding derived strength or stiffness parameter should have an associated reliability function. Through this level of information, the geotechnical designer and the end-user should be able to gauge the level of confidence they can have in the digital twin and can carry out sensitivity analysis to fully mitigate any risks associated with the relevant uncertainty and plan for future expansion. As suggested by Phoon (2017), reliability methods can handle complex real-world information (cross and/or spatially correlated multivariate data) and information imperfections (sparse, uncertain and/or incomplete information) more effectively than relying on empiricism and judgement alone. The principle of adopting reliability function is not new, but for use of digital twin in geotechnical engineering, it assumes more importance and any digital tools shall prompt the modellers to ensure this information is made available.

3.3 Quality and Quantity of Data

It is a well-known fact that timely and correct decisions on construction projects can be made when the right level of information is provided at the right time to the right person.

The geotechnical industry has long been using digital data formats to capture ground investigation data. However, the data mostly remains of factual nature. Mostly on large infrastructure projects too much data is available that doesn't match the expectations of the geotechnical designer, as it is either not available in the right format and at the right quantity, or it is provided too late and doesn't match the project's programme requirements.

UK's AGSi, AGS-piling and AGS Instrumentation and Monitoring seem to be the right way forward to assist with the design process. However, its use has yet to get traction as an industrial practice internationally.

When it comes to automation of the interpretation process of digitally available data, geotechnical engineers feel nervous to let its full implementation happen, as they are aware of the complexity of the process and thus the need for higher level of human intervention. As an example, while reviewing plots of SPT, CPT and other geotechnical in situ and laboratory testing data, the reasons for outliers in the data are not always clear. It could be a naturally occurring localised pocket of soil or rock that might be presenting completely different strength or stiffness characteristics, or it could be a damage caused to the in-situ material through the drilling and testing process, or even disturbance to the laboratory samples during transportation and preparation. Skill of the drilling crew and the methodologies they adopt on site are known to result in highly variable results as can be seen in two RQD results obtained from adjacent boreholes across a reasonably uniform site (Figure 5). The boreholes were carried out by two different ground investigation contractors. Only human intervention in the automation process can make it possible to recognise any such systemic risk that results in anomalies in the available geotechnical data, and hence avoid a potential negative impact on the design outcome.

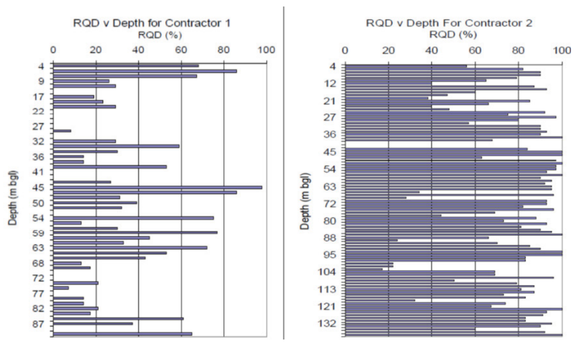

Figure 5. Drilling Induced difference in RQD results

Similarly, interpretation of geological stratigraphy from boreholes is widely carried out by commercially available software; however, final geological model for design purposes requires human intervention to identify features such as faults, buried river channels, historic fill and buried utilities or structures. Nevertheless, case studies exist where comparison of interpretation carried out of insitu test results such as CPTs by AI based digital tools and those made by humans suggested CPT interpretation required relatively fewer corrections after review (Kennedy, 2021).

Safe design requires sufficient and good quality data. It is usual for geotechnical engineers to criticize the scarce data available for design. Limitations in data are usually complimented by experience and judgment. The level of uncertainty or the quantum of perceived risk by a geotechnical engineer in relation to relying on a particular data set cannot always be explicitly conveyed in the design reports or calculations, regardless of whatever the design claims to have achieved as the 'nominated' factor of safety based on standard or codes. For example, geotechnical engineers consider a multitude of factors while selecting a 'design line' based on a given data plot of soil or rock parameters. These might include non-quantifiable risks or factors such as the level of experience the supervision staff had (if there was one present at the time of drilling and testing), skill level of the drillers or the testing team, techniques used to obtain the data, any potential disturbance, quantity of data and sensitivity of the structure being designed to the design parameter in question. Accordingly, the process of selecting the so-called representative, worst credible, spatial mean, local mean or characteristic design parameters can be subjective. The result is that geotechnical engineers are often criticized for producing designs which are supposedly overly conservative or overly safe in the eyes of the party who are bearing the cost of construction, as the uncertainties are not always explicitly conveyed merely through the adopted 'nominated' factor of safety. Yet another issue arises when in some cases there is too much good quality data available, but the conventional interpretation process does not allow any benefit of the abundant good quality data.

Merely digitising the data capture and management process might speed up the overall interpretation process if conventional interpretation rules are followed, but it will continue to miss out on recognising any discrepancy or even benefit caused by the above factors which are mostly related to systemic risks (or opportunities) rather than statistical risks. Use of a reliability approach could potentially help with alleviating some of the issues related to data scarcity as well as provide better results in the situation where data is abundantly available.

In case where data is too scarce, more generic data can be used instead of site-specific data. The main challenges in use of generic data to supplement project specific data are possibly jurisdiction's requirements to share geological data, as well its accuracy and reliability. This also warrants human intervention to consider two different sets of data having a difference in 'confidence level'. These factors can, however, be built-in into the reliability analysis so that both data scarcity and data abundance can be weighted into the interpretation process more objectively.

3.4 Ownership of Data

The nature and size of today's infrastructure projects often require the consultant to work in partnership with joint ventures, alliances or consortiums, which can raise issues regarding the ownership of the project's data. Development of a comprehensive Digital Data Management Plan should be undertaken during the tendering phase to identify these issues. As a minimum, such a plan should agree on a common data environment which will provide a single truth for the development of the federated digital model; agreement on the common data environment for the storage and display of the model (together with associated drawings and reports developed on the back of the models) and the workflow process to ensure the current model is shared between the parties. Such a plan should address the ownership and distribution of the data, but it is important that consideration is also provided to addressing the intellectual property used in developing the digital model.

An example recently has occurred on a joint venture project that Aurecon was part of where the logs and laboratory testing results have been provided from a site investigation partner in an AGS format, however as the interpretation of the geotechnical domain was being undertaken by the joint venture partners, additional functions had to be amended to the data set to allow grouping and characterisation of the data into each of the domains.

3.5 Nature and Life Cycle of Geotechnical Risks

Geotechnical risk management is a highly complex area which requires human intervention at all stages of a project. These risks remain live throughout the project life cycle, starting from bid phase and throughout the design life and de-commissioning phase of the asset.

The process of geotechnical risk identification can be highly subjective, and perception of risks does vary depending on the contractual party or stakeholder identifying or assessing risks. For instance, depending on the procurement type and contractual clauses such as whether a differing ground conditions clause is added, the client's perception of severity of risks can be different to that of the geotechnical consultant or contractor.

Traditionally, geotechnical risks on projects have mostly been assessed via a qualitative approach. A qualitative approach measures the impact and likelihood of the identified risks and ranks the risks in a risk register for mitigation analysis. The qualitative approach mostly used simple excel spreadsheets at most and has hardly gained any noteworthy benefits from the available digital technology.

The quantitative approach is a more involved process compared with the qualitative approach. This approach is usually reserved for larger, more complex projects. Although this approach normally adopts a non-simulation approach using a single variable or factor, recent digital advances have facilitated the use of simulation approach, such as Monte Carlo Simulation, which allows to calculate the combined effect of several geotechnical risk factors. Detailed descriptions of digital simulation methods are in US DOT manuals and other published books and reports. However, to date, the simulation approach is carried out at the overall project level and not exclusively used for geotechnical risk assessment. Further, mostly aleatory, or statistical uncertainties are captured and not the systemic risks, which is a major gap area to be considered in future work. NCHRP Guidelines for Managing Geotechnical Risks in Design Build Projects (Gransberg et al., 2018) does provide preliminary level guidelines to the simulation approach, which can improve the state of the art and more widespread use of digital simulation approach for geotechnical risks assessment.

3.6 Interoperability of Digital Design Tools

In Digital Geotechnics, much of the recent development appears to be heading towards a Virtual Reality end use (Output) or the Data Capture (Input) Stages. In the last couple of decades, many newer software tools have appeared in the market with significant improvement in processing speed and yet analysis time hasn't significantly reduced, mainly due to lack of interoperability among the software tools at various stages of the geotechnical design process.

Most available digital design software tools remain to be operating in a 2D plane strain or axisymmetric domain which are limited in their representation and assessment of a real-world 3D failure mechanisms and ground response. A handful of 3D design software is available; however, these are only used for complex problem solving as the generally acceptable program and cost expectations by the clients do not permit a more widespread use of these tools. Such design software tools mainly rely upon finite element or finite difference routines and although computing speed has significantly increased in the last two decades, lack of interoperability of these digital design tools continue to require significant skilled labour in frequently utilising them for geotechnical design.

Over the last few years, several software developers have worked to enable their software to directly import data from 2D and 3D digital geological models into analysis software however, these functions have yet to become a general practice as they still require human intervention and labour to avoid significant errors and omissions. Further, once analysis is carried out and a solution is developed, there is yet another phase of manual export of information to regenerate the final solution in a 2D or 3D digital model for the purposes of visualisation and production of construction drawings.

3.7 Role of Human Judgement in Design

Judgement is the ability to arrive at sensible decisions about a problem in the presence of incomplete and contradicting information (Jose, 2021). Judgement is the exercise of thinking clearly, logically, and calmly about a problem, weighing the known facts, suppositions, missing information, and consequences, and then taking a decision (Marr, 2006).

Although judgement plays a central role in all phases of geotechnical engineering process, interpretation of geotechnical data for design purposes is the prime area where digital transformation is facing the most resistance out of all other areas.

International geotechnical design codes and standards such as the Eurocode 7 have attempted to implement the same principles of reliability analysis as have been successfully adopted by their counterpart structural design codes for instance, however, they do recognise the role of human judgment in geotechnical design, and thus Eurocode 7 allows the designer a significant degree of leeway when it comes to use of judgement in the selection of characteristic parameters for design purposes. Similarly, the partial factors of safety, as stated in Eurocode 7, are yet to be explicitly calibrated according to reliability principles described in Annex C of the head Eurocode (EN 1990:2002) or Annex D of ISO2394: 2015 (Phoon, 2019).

Variation in the selection of geotechnical parameters could be due to actual natural variability, inherent bias of the sub sampling for testing or due to human limitations of testing and predicting the actual soil or rock characteristics. In addition, the available design methodologies are less than perfect and in fact mostly empirical as principles of exact science haven't yet developed constitutive relationships that fully model the vastly variable behaviour of soils and rock and their interaction with structures under different levels and forms of loading. Numerous constitutive models exist however, selection of the most appropriate model for a specific design purpose requires significant experience and judgement on behest of a geotechnical engineer and cannot be merely left for the machine to decide upon. Whether a plane strain 2D model, an axisymmetric or a more time-consuming (and costly) 3D model is appropriate for a given problem, experience and judgement are crucial.

Even for the so-called simplest of geotechnical design problems such as estimating pile lengths and predicting settlement of an isolated footing, case studies suggest a significant variation might exist among the results produced by different geotechnical engineers. In one case, prediction of pile lengths by sixteen geotechnical engineers all resulted in pile capacity varying by 50% to 200%, when compared with the actual capacity resulted from a pile test on site (Figure 6, Wheeler 1999). Similarly, routine calculations of the settlement of footings on sand can easily be in error by a factor of 5 (Figure 7, Clayton et al 1988).

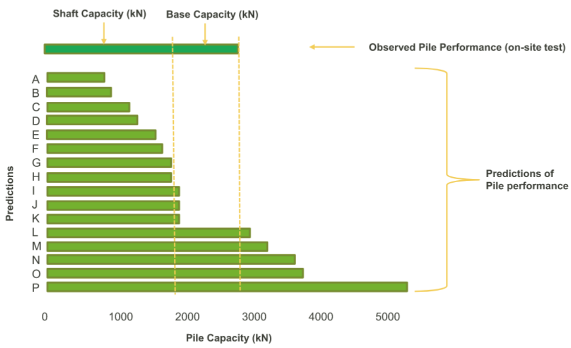

Figure 6. Inaccuracies in Pile Capacity Predictions (Wheeler 1999)

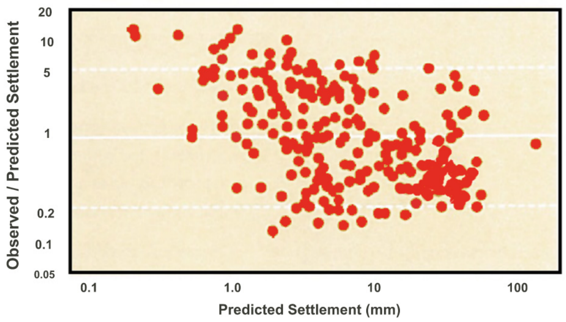

Figure 7. Inaccuracies in Prediction of Settlement (Clayton et al 1988)

Despite the undeniable role that judgement continues to play in seeking geotechnical solutions, its role has indeed evolved over the last century from a more gut feel type approach based on limited and rather crude testing data, to a more informed level of judgement supported by better field and in-situ testing; and analytical, numerical and probabilistic analysis models, as depicted in Figure 8 (Lacasse 1988, Mayne 2012). This is further supplemented by ease of access to digitally available information related to lessons learnt from others. The immense power of computing and digitisation can be exploited by using the available information in a more systematic and transparent manner to support engineering judgement. To speed up the transformation process, digital workflow for geotechnical design will need to be modelled such that stages for human intervention are well defined and so that the geotechnical engineer is prompted, where their input is needed.

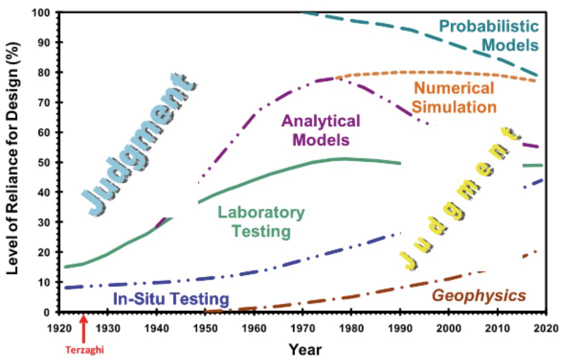

Figure 8. Role of Judgement in Geotechnical Engineering (after Lacasse 1988, as referenced in Mayne 2012)

4 CONCLUSION

Implementation of digital transformation in geotechnical engineering is creating benefits in terms of time savings, cost reduction and transparency to geotechnical design and risk management processes throughout the life cycle of a project. However, it is crucial to understand the limitations and challenges that this transformation engenders, since geotechnical engineering, unlike some of the other Civil Engineering fields, deals with enormous complexity and uncertainty associated with naturally occurring materials. As a result, digital formulations would need to formally adopt and imbed empirical methodologies and human judgement as part and parcel of the geotechnical engineering process.

A systematic and transparent framework is required to implement digital technologies in this field so that some of the concerns of the geotechnical engineers and the broader engineering community can be alleviated. A key challenge is to achieve transformation holistically and not in a particular challenge area to avoid chaos and disruption in the industry. All the challenges associated with digital transformation, such as those related with integration of human judgement at the right stages, holistic risk management, interoperability among digital tools, a shift to reliability approach in design, staff upskilling and investment in digital tools; all need to be tackled together rather than in isolation. This can be achieved by developing digital workflows that clearly highlight the stages and gateways where human intervention is required and also by identifying areas where further rapid transformation can be instigated in the field for the ultimate benefit of the construction industry and society as a whole.

5 ACKNOWLEDGEMENTS

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Digital optimisation workflow in early project phases and what it can bring when looking at the MacLeamy curve

L. Nguyen[1]

[1]Geotechnical design & development engineer, Bauer Spezialtiefbau GmbH, BAUER-Str. 1, 86529 Schrobenhausen, Germany; PH +49 8252 97-2370; email: luan.nguyen@bauer.de

ABSTRACT

It is known from the MacLeamy curve that early effort in project developments pays off. Any change in early project phases takes less effort and is more effective in impacting cost and success of the project than a change that incurs later. This contribution points out possible savings by employing numerical optimisation for achieving the optimal design with respect to both code-conforming performance and construction costs for common geotechnical systems such as ground improvements. One of the bottlenecks for the widespread use of numerical optimisation for design is perhaps the lack of a workflow management program. In the present paper, a structure of a functional workflow management program based on Python, its standard library, third-party packages, and external APIs will be detailed. In addition to that, some fundamentals of how optimisation works will be presented. One meaningful use case of the automated optimisation applied to the design of ground improvements for LNG tank foundation is presented. Extensions to digital ground model provided by the BIM process and seamless information transfer to the construction site purposed for the implementation of the observational method in geotechnical design will be drawn.

Keywords: geotechnical design, numerical optimisation, digital workflow management, productivity

1 INTRODUCTION

Although a lot of innovations and great improvement effort, the AEC industry is still faced with low figures in either productivity or sustainability indicators. The McKinsey Institute reported an averaged 1 percent labour-productivity growth in construction over the past two decades, compared with 2.8 percent growth of the total world economy and 3.6 percent growth in manufacturing sector (Barbosa et al. 2017). Being one of the largest industrial sectors this poor development in productivity has caused great economic loss and adversely impacted resource efficiency and sustainability.

The above-mentioned McKinsey report drew out 7 measures to increase productivity in construction, among them rethinking design and engineering processes and leveraging digital technology, new construction materials and advanced automation. What was not mentioned as a measure to increase productivity in construction is the use of computerized optimisation methods. Although numerical optimisation as a sub-field of applied mathematics is the backbone in many scientific research and engineering fields, its use in geotechnical design practice has been very limited to date, at least to the author's knowledge while working in the topic for several years.

Aside technical enhancements and the adoption of advanced technologies, new ways of collaboration and information integration have been introduced to reduce fragmentation and backwardness in design/ build/ manage processes (MacLeamy, 2004). The MacLeamy time-effort curves (Figure 1) are often used in AEC to draw the effort or effect of changes with respect to time in the realisation of a construction project.

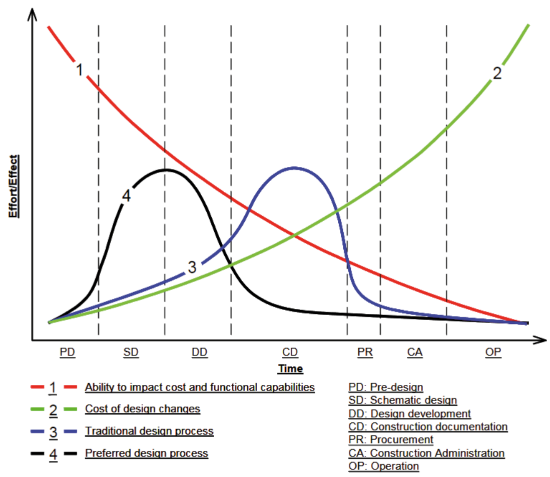

Figure 1. The MacLeamy time-effort curves (MacLeamy, 2004)

In Figure 1, curve (2) is self-explained: a design change at later project phases cost incurs more cost than a change that happens earlier. The opposite is true for curve (1): the ability to impact cost. A traditional design and construction process will likely result in curve (3) where high cost and effort are required by late changes. A preferred curve is curve (4): impactful changes are made early at low cost. The MacLeamy theoretical curves imply that expenditure at early project phases will pay off quickly by saving unnecessary changes that usually occur lately. The AEC started to adopt digital building information modelling (BIM) which is considered as one of the measures to bring the project execution from curve (3) to the one that is closer to curve (4).

It is noted that ground data and their uncertainty exist as they are provided by standard soil investigations and geotechnical laboratory tests. Design optimisation offers no solution to reduce uncertainty either in spatial ground variability or in the values of geotechnical parameters. Ground uncertainties should rather be handled by the selected design code, for example, with using partial safety factors in design. Some design codes, such as Eurocode 7 (EN 1997) even advocate the use the observational method in design in the face of high uncertainty.

It must be stated that the amount of ground data can increase during project development phases if further soil investigations and laboratory tests are further requested. The uncertainty associated with ground data can reduce as a result. This condition is not ideal for the design approach that is followed here. The deployment of design optimisation is therefore most meaningful if a rather complete soil investigation campaigns have been carried out. Early project phases therefore mean from this time.

This paper advocates for more expenditure and effort in the design stage to achieve the optimized solution for a geotechnical design problem. In support of a seamless integration of optimisation in design, the following requirements are to be made available:

- a detailed 3D ground model,
- a sophisticated calculation program that well captures geo-mechanical behaviour and soil-structure interactions of the design problem, and
- a workflow management program with numerical optimisation method.

In early geotechnical design, usually a very simplified 2D geological model is considered from information of the bore logs. Similarly, soil and rock parameters are only roughly estimated, allowing the geotechnical designer to deliver the design solution very soon so that a project cost can be quickly estimated by the tender engineer. The flip side this process is 1) any mismatch or misinterpretation of the ground model and soil/ rock parameters will then change the design significantly and 2) the design is often far conservative or not optimal.

Given the same amount of soil investigation data, however, additional information can be gained for design. In particular, more often than not, a 3D ground model can be built if some extra time budget is allowed. If the BIM process is adopted from the beginning, the 3D ground model is preferably extracted from the BIM model. Here, machine learning methods for soil type classification based on in-situ CPTs can be used (Rauter and Tschuchnigg 2021). An extension to automated soil parameter determination and building entire 3D underground models, which serves finite element and BIM model building, will greatly benefit the geotechnical design engineers (Brinkgreve and Brasile 2022). In the case detailed oedometer and triaxial laboratory tests are available, an automatic constitutive model calibration scheme can also be carried out for getting parameters of advanced constitutive models, such as for the hypoplastic soil model (Machaček et al. 2022).

The second requirement is relatively easily achieved today as many proprietary geotechnical software providers implement state-of-the-art sophisticated soil models based on plasticity or critical state soil mechanics theories, and structure and soil-structure interactions under monotonic/ cyclic loadings in drained/ undrained conditions. Many geotechnical construction/ consultancy companies and research institutes have also developed their own advanced computation code for specific geotechnical design tasks, which can also be suitable for design optimisation. It remains for the geotechnical designer to create an adequate calculation model using soil parameters from soil investigations, laboratory tests, or proper correlations.

The third requirement is often not ready. The reasons can be 1) optimisation is not needed in a traditional design approach and 2) design software providers usually do not see the potential to optimize or do not have insight in how much benefit optimisation can bring while geotechnical design engineers working with a consultancy or construction company are often not involved in extensive coding nor have reserved time and budget to do so.

In the following, I will show that such a workflow management program can be developed using Python, its standard packages, and any external APIs provided by a proprietary/ in-house software. A use case for the optimisation of ground improvement is then provided to showcase the power of numerical optimisation for geotechnical design.

2 WORKFLOW MANAGEMENT

2.1 Implementation

It is a conception that implementation of such workflow programs is the job of PhD students and researchers who have plenty of time to mock-up such a complex workflow management program to test old and new algorithms on a benchmark problem. This is not true anymore with today's array of development tools, machine learning and optimisation algorithms one can reach at the fingertips from public domain. But as for geotechnical design, one should understand how soil, rock, and structural elements behave to carry out geotechnical design, to leverage the power of an algorithm, one should understand why and how it works. There are plenty of books and short courses available in the object for self-learning if one has not attended such a class at university. After having some basics, one can practise implementing from scratch one of the optimisation algorithms of preference or one can also pull the source code from a good implementation stored in Github repository for practice. While coding such a program in Python, the core libraries in Python and third-party packages, such as numpy, scipy, pandas, matplotlib, already provide sufficient tools for numerical calculations and visualisation.

For a rigid implementation of an automation workflow, it is very beneficial to organize the program in an object-oriented programming paradigm. Figure 2 displays such a workflow for geotechnical optimisation which task is mainly to plug a solver of choice in a selected optimisation algorithm. It is noted that many software design patterns can be used to build the workflow's components and connections that are easy to expand and maintain.

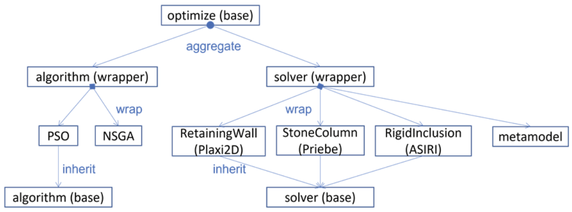

Figure 2. Object-oriented organisation for design optimisation workflow

It is noted that interfaces to an external calculation such as Plaxis2D/ Plaxis3D solver can be quickly built by using the API provided by the software.

If the calculation model is computationally intensive, such as a large finite element or finite difference model, building a surrogate model, also called meta-model, can be very beneficial. The idea behind meta-model is to approximate the original computationally heavy model by a lightweight reduced order model. Such a reduced model can be trained from a set of sampled data points generated by the original model by using, for example, a multi-layer artificial neural network as used for back back-calculating soil parameters in tunnelling (Nguyen and Nestorovic 2013) or by using a Gaussian process (GP) as presented in (Nguyen and Haddad 2022) for the design of foundations and retaining walls. The process of meta-model building requiring model training and model validation steps is illustrated in Figure 3. It is noted that the once the meta-model is built, it can be deployed for various purposes, such as sensitivity analysis, back analysis, and design optimisation.

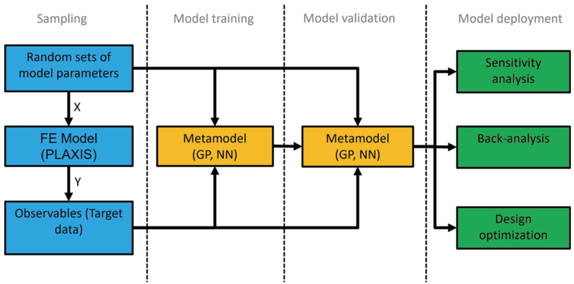

Figure 3. Meta-model building process

2.2 Multi-objective optimisation for geotechnical design

Multi-objective optimisation (MOO) has been used in structural designs in manufacturing engineering, for example, as an engine for generative design. MOO can be readily used for optimizing geotechnical design as well.

A geotechnical design problem is most often multi-objective rather than single-objective in which the design objectives compete each other. For example, in the design of pile-group foundation or ground improvement, it is of interest to minimize the total concrete volume and reinforcement amounts (monetary cost) and to maximize the serviceability performance, such as the minimisation of settlements.

Multi-objective optimisation methods most often belong to the family of meta-heuristic optimisation methods. An effective population based meta-heuristic optimisation methods used for multi-objective optimisation is the non-dominated sorting genetic algorithm 2 (NSGA-II) (Deb et al. 2022). One fundamental mechanism for the algorithm to work is the ability to randomize candidate solutions, which helps the algorithm to explore the design possibilities. The coded gene evolution mechanisms such as mutation, cross-over, and selection make sure that diversity and survival of the best solutions are maintained in the optimisation process. One big advantage of the multi-optimisation approach is that it results not only one design solution, but also a range of optimal solutions that can be selected based on certain preference such as cost or settlement requirement.

Early introduction of MOO to foundation design includes the work of Kinzler et al. 2007 in which the authors proposed to use the optimisation method to the design of pile groups. Recently, Shen et al. 2022 proposed a generative design framework supported by meta-modelling for the design of wind turbine foundations. The NSGA-II is perhaps most often used for multi-objective optimisation in engineering. The newly introduced Generative Design module in (Autodesk Revit 2021) is told to use NSGA-II optimisation in its implementation for multi-objective optimisation.

As soon as the computational model has been created for the geotechnical problem at hand, the computational model can be parametrized with the selected design parameters. The choice of design parameters can be intuitively decided by the design engineer or by performing a thorough sensitivity analysis. In design of geotechnical structures, there are often a few parameters to be considered. For example, the most important design parameters for ground improvement by reinforcements are the size or diameter of the reinforcement's cross-section, the depth of reinforcements, and the grid spacing between reinforced elements. The process to arrive at a set of design parameters that works often requires repetitive trials that are manually carried out in the design process. Because of the complex load transfer mechanism between surrounding soil and reinforcements, iterative tuning is required to get to a satisfactory result, which does not necessarily mean the best solution that can be drawn for the problem at hand.

We, however, propose to use a numerical algorithm in the search for the optimal design solutions. Assume that three design variables x_1, x_2, and x_3 are selected for the design optimisation problem at hand. The optimisation problem is formulated as:

$$\text{find: } x_1, x_2, x_3 \quad (1a)$$

$$\text{such that: } f_1() \text{ and } f_2() \text{ are minimized,} \quad (1b)$$

$$\text{while being subject to: } g_1(), \quad (1c)$$

where $f_1()$, $f_2()$ are objective functions, $g_1()$ a constraint function. Both objectives and constraints are functions of the selected design variables. It is noted that there can be more than two objectives and additional constraint rules in a design optimisation problem. An objective function is a function that defines the intended goal for the optimisation algorithm to minimize or maximize, for example settlement and construction cost. The constraint rules, on the other hand, will guide the optimisation algorithm to skip candidate solutions that violate certain project defined rules or code based and construction related constraints, for example the column grid spacing should not be less than a certain ratio to the column diameter.

3 A USE CASE

Ground improvements by installing reinforcement elements into the ground provide economic solutions over foundation piles in applications, such as foundations for structures of moderate loads such as warehouse, water or LNG tanks, etc. By adding reinforced elements into loose or soft soils, stiffness and bearing capacity of the composite soil-structure system are increased, allowing to reduce settlement, susceptibility to shear failure, and even liquefaction potential.

The design of ground improvements often consists of iterative trials of various reinforcement configurations. Especially when purposed to work as floating foundations, the choice of reinforcement diameter, depth, and spacing become hard to decide without iterative trials as one design parameter influences the others. Most often, under a strict deadline for design delivery, a geotechnical designer will have a preference for a configuration of design parameters that works with respect to the ultimate and serviceability limits. Although achieving a design that works is good, the fact that the cost of the design solution can further be optimized is not further pursued makes overdesign a source of inefficiency.

The little effort spent in this early design likely leads to a suboptimal design, which may need substantial design change later. For a contractor, this means higher bidding price and therefore lower chance of getting the project granted. For the owner, it can be unbeneficial, especially if a kind of lump sum contract was signed based on the initial design. Looking back at the MacLeamy curve in Figure 1 at this point, one can see that those cumbersome issues are best solved by spending effort at the early design phase. Running a thorough optimisation campaign using numerical optimisation in the early design phase can help to minimize the additional costs caused by late substantial design changes.

The question many engineers, consultants, and owners ask is: can a geotechnical design solution that works also be an optimal solution? State-of-the-art mathematical optimisation provides the answer to it. Nguyen and Haddad 2022 showed that numerical optimisation can help achieve a set of optimal solutions for the design of ground improvements and retaining walls. A standard procedure for design optimisation of ground improvements, e.g., for tank foundations, is presented in Figure 4. Compared with a conventional design approach, one needs to set up the design problem as an optimisation problem with design variables, objectives, and constraint rules and let the automated iterative optimisation process run on a computer.

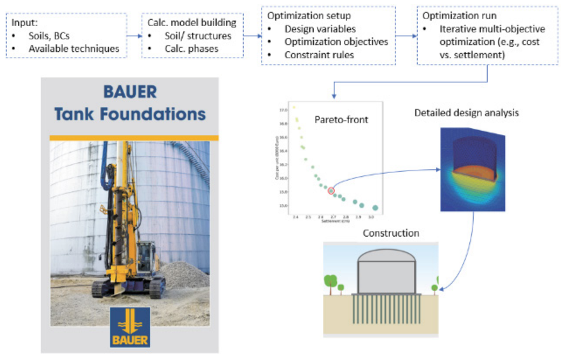

Figure 4. Optimisation workflow for tank foundation by rigid inclusions

In the example here, a rigid inclusions design for an oil tank is optimized using a multi-objective optimisation algorithm NSGA-II. Working surface load of the tank is 155 kPa. The soil profile represents various layers of very soft clays and loose sands that are alternately present until the depth of 25 m (Figure 5).

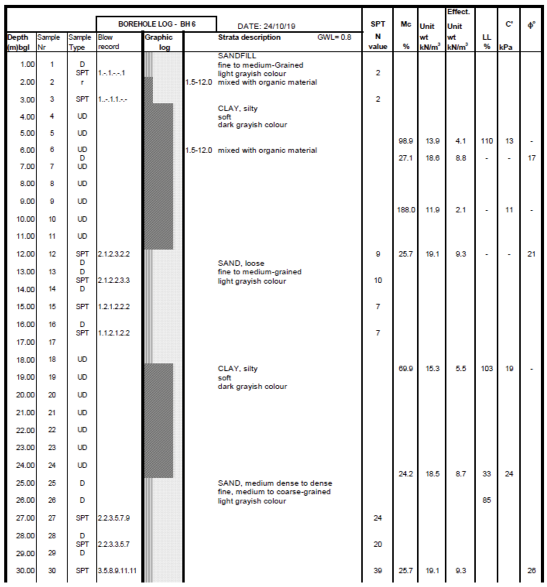

Figure 5. Soil profile needing rigid inclusion foundation for oil tanks

The optimisation problem is set out with using the full displacement columns of diameter 44 cm as rigid inclusions. To keep a practical replacement ratio, allowable grid spacing is limited in the range [1.8 m, 3.6 m]. Considering weak soils in the first 25 m, the allowable column length is set in the range [25 m, 45 m]. The two objectives for optimisation are

- Minimisation of the surface settlement in serviceability. The settlement is calculated following the ASIRI method (IREX 2017).
- Minimisation of cost. The cost function is simply proportional to the concrete volume with a base unit price.

Results of optimisation run clearly show convergence towards the so-called Pareto-optimal solutions. The optimal design solutions achieved after 20 iterations are displayed in Figure 6. It can be seen in the variable space that column lengths are just enough for the column to be embedded in the competent sand, where tip resistance is given. Grid spacings of the column raster range from 1.7 m to 2.8 m, providing a range of settlements with correspondingly construction costs.

A remarkable outcome of the optimisation run is that it results in a pool of optimal design solutions. Then a preferred design solution can be chosen for submission depending on the settlement requirements and/ or the client's budget. The impact of allowing for more settlement on saving in construction costs can also clearly be presented for the client and engineers for discussion and decision making.

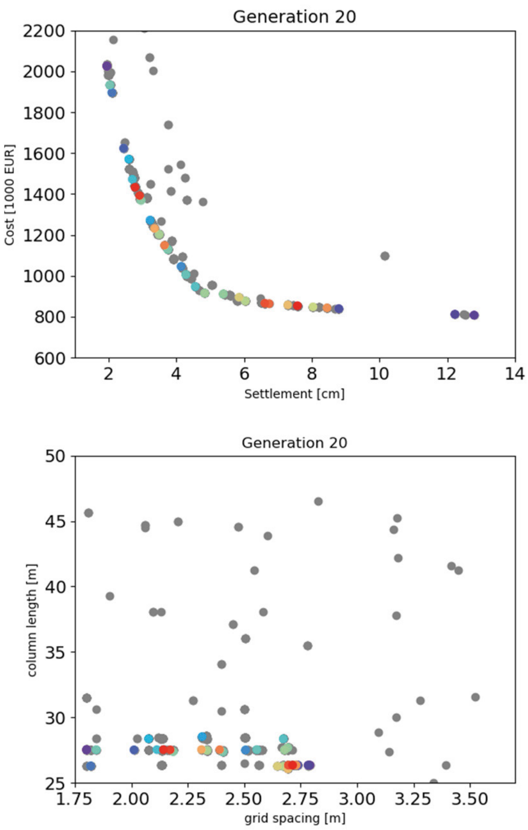

Figure 6. Optimisation results. The points in color are design solution converged at the 20th generation of NSGA-II run. The points in grey are points at previous generations.

Transfer optimized design to a sophisticated finite element (FE) model for detailed geomechanical analysis can be readily automated by using a Python scripting that interfaces with the FE program's API.
The FE model shown in Figure 7 displays results of settlement analysis for one of the optimal design solutions shown in Figure 6.

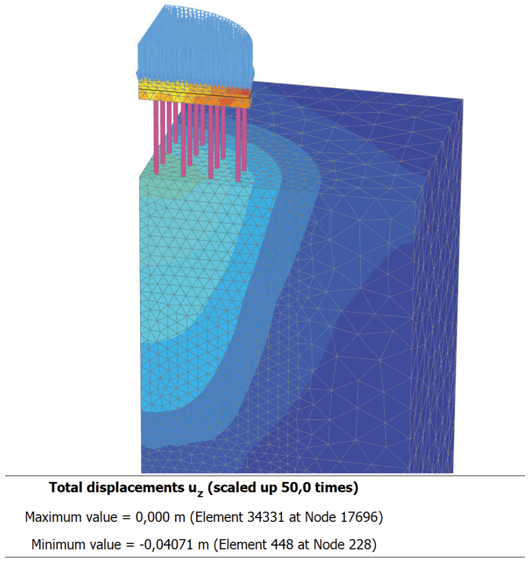

Figure 7. Transfer of the optimized design parameters to a 3D finite element model for detailed analysis

In the same way, any design in the pool of the optimal designs can be conveniently linked to a BIM software such as Revit for effective presentation and discussion with the client and consultant. Figure 8 displays in Revit, for example, one among 20 optimal designs seen in Figure 6. The information in the BIM model serves as a basis for construction work and semi-automated quality control during and after construction.

This connection between design and 3D visualisation is best carried out by a compact automated workflow such as using the visual scripting language Dynamo (Dynamo BIM). An open BIM format or a proprietary BIM software such as Revit offers a convenient method to realize this task. Interoperability is key to stakeholders, which enables data transfer among various software tools, helping to eliminate data silos that often occur in realisation of a construction project.

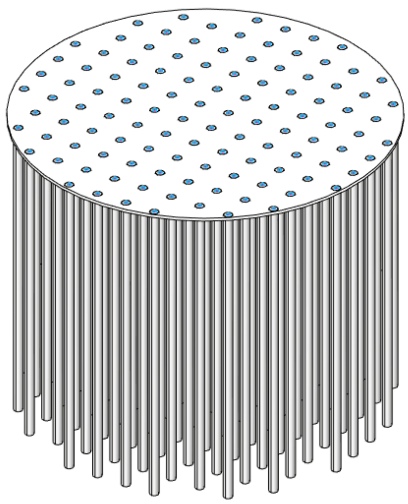

Figure 8. Transfer of the optimized design for visualisation and inspection in a BIM model

The integration of IoT sensors installed at the construction site is a further step forward. Easy and reliable access to monitored data at the construction site will facilitate the implementation of the observational method (OM) in geotechnical design (Peck 1969, Powderham and O'Brien 2020). OM in geotechnical design can be implemented using data such as wall deformations and surface settlements received from continuous monitoring with IoT sensors. OM will help the geotechnical designer, for example, to quickly modify the design of the supporting struts or ground anchors given the actual ground and groundwater conditions encountered at site. In the case of challenging excavation, OM assists in predicting stability of the excavation pit by providing the designer with the most updated geotechnical parameters obtained from performing back analyses. OM ultimately corrects the initial design based on information from the soil investigation, which can be conservative or incorrect, toward a design based on actual soil conditions and parameters found on the ongoing construction site.

4 CONCLUSION

Heavy exploration in the design parameter space at the early design stage is made possible using a multi-objective optimisation as presented in this paper. This early upfront effort, which is readily orchestrated with the help of a workflow automation program, can help to achieve fine-tuned optimal design solutions right in the first design round. Value engineering can well work in tandem with optimisation workflow to come with the most suitable geotechnical system among many comparable techniques, for example pile foundation vs ground improvement and secant piled wall vs diaphragm wall and set up the most meaningful scenarios to be optimized.

The optimal design solutions can then be transferred to a sophisticated 3D visualisation and simulation model for further detailed analysis and discussions. This step is preferred carried out automatically for speed and accuracy.

The integration of IoT sensors' data provides a good platform for the implementation of the observational method in geotechnical design, which helps to reduce uncertainty in geotechnical parameters and therefore increases confidence in design.

All the measures ultimately help to bend the MacLeamy curve of each project to the one closer to the preferred curve (4) in Figure 1.

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Digitalisation and Automation of Road Materials Compaction: SPARC Intelligent Compaction Kit

A. Sounthararajah[1], S. Sivagnanasuntharam[2], R. Ranasinghe[3] and J. Kodikara[4]

[1]Research Manager, ARC Industrial Transformation Research Hub (ITRH) – SPARC Hub, Dept. of Civil Engineering, Monash University, Clayton Campus, VIC 3800, Australia; email: arooran.sounthararajah@monash.edu
[2]PhD Scholar, ARC Industrial Transformation Research Hub (ITRH) – SPARC Hub, Dept. of Civil Engineering, Monash University, Clayton Campus, VIC 3800, Australia; email: suthakaran.sivagnanasuntharam@monash.edu
[3]Research Engineer, ARC Industrial Transformation Research Hub (ITRH) – SPARC Hub, Dept. of Civil Engineering, Monash University, Clayton Campus, VIC 3800, Australia; email: rajitha.ranasinghe@monash.edu
[4]Professor and Director, ARC Industrial Transformation Research Hub (ITRH) – SPARC Hub, Dept. of Civil Engineering, Monash University, Clayton Campus, VIC 3800, Australia; email: jayantha.kodikara@monash.edu

ABSTRACT

'Intelligent Compaction' (IC) broadly refers to the compaction of road materials, primarily using advanced sensing and automation, which achieves the target performance over their design life. Our recent international workshop on intelligent compaction highlighted that countries like US and China have implemented IC technology in practice as a mandatory requirement for contractors almost 6 years ahead of Australia. Our online questionnaire survey results indicated that the slow adoption of IC technology in Australia is mainly due to the lack of standards or specifications for the use of IC technology and the lack of confidence among contractors who already have an existing fleet of conventional rollers for compaction. There are some retrofittable kits available in the market that can facilitate IC with conventional rollers. However, the main limitation of these kits is that they provide only one parameter out of various intelligent compaction meter values (ICMVs). We are developing an innovative kit with cutting-edge hardware and software tools to facilitate performance-based compaction of road materials. The key features of our kit include [i] facilitating simultaneous visualisation of multiple ICMVs on both onboard and remote systems in real-time during compaction, [ii] providing versatility to retrofit a conventional roller, [iii] flexibility to incorporate corrections for different ICMV indicators, [iv] facilitates customising to construction specifications in line with the ongoing industrial digitalisation, and [v] integrable with the existing post-processing software such as Veta to view and analyse the collected IC data. In this paper, we provide the basic design concepts of the kit, its functionalities and capabilities with initial test results. The design concepts of the kit prototype will be further refined in the future based on the field trials undertaken on different materials using different roller types. The experiences gained through using our kit in actual construction projects will pave the way to develop robust and data-oriented specifications for IC to be used by the Australian road construction industry.

Keywords: intelligent compaction, road pavements, rollers, intelligent compaction measurement value, retrofit kit, performance-based specifications

1 INTRODUCTION

Road pavement materials are traditionally compacted using conventional rollers, and post-compaction spot tests are performed to assess the compaction quality and uniformity. Spot tests include a nuclear moisture-density gauge (NDG) for density and moisture measurements, lightweight deflectometer (LWD) for modulus measurement, static plate loading tests (PLT) for stiffness measurement, and sand replacement method for density measurement. The main drawback of these measurements is that they do not represent the entire compaction area. The intelligent compaction (IC) technology fills this gap by measuring material response or compaction level continuously in real-time with an instrumented vibratory roller and varying the frequency and amplitude of the roller automatically to optimise the compaction process. The technology provides the roller operator with real-time continuous feedback of the level of compaction through colour-coded maps, which helps prevent under compaction or over compaction for the entire compaction area. Figure 1 shows schematically the sensors and their locations in an IC roller.

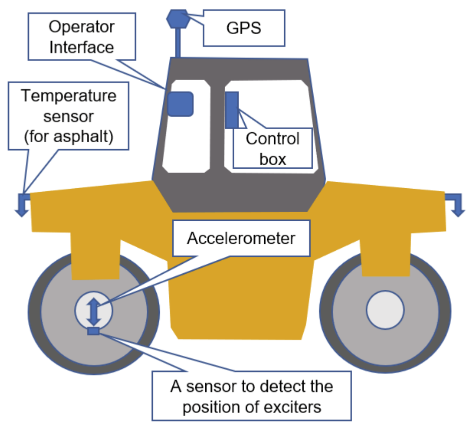

Figure 1. Schematic diagram of an intelligent compaction (IC) roller [adapted from Sivagnanasuntharam et al. (2021)]

A handful of roller manufacturers are currently making specialised drum rollers with built-in IC technology, such as BOMAG and Dynapac (Savan et al., 2015). The recent international workshop on intelligent compaction organised by SPARC (Smart

Pavements Australia Research Collaboration) Hub highlighted that countries like US and China have implemented IC technology in practice as a mandatory requirement for contractors almost 6 years ahead of Australia. The slow adoption of IC technology in Australia could be due to the lack of confidence among contractors who already have an existing fleet of conventional rollers for the compaction of geomaterials. There are some retrofittable kits available in the market that can facilitate IC with conventional rollers. However, the main limitations of these kits are that they provide only one parameter out of various intelligent compaction meter values (ICMVs) and they do not record the raw data collected from sensors, such as accelerometers.

The SPARC Hub IC team develops an innovative IC kit with cutting-edge hardware and software tools to address the above issues. The prototype of the kit is built from scratch and is designed to handle multiple ICMV parameters within the limits of retrofitting capabilities. The key features of the current version of the SPARC IC kit include:

[i] facilitates simultaneous visualisation of multiple ICMVs on both onboard and remote systems in real time during compaction;
[ii] provides versatility to retrofit a conventional roller;
[iii] flexibility to incorporate corrections for different ICMV indicators;
[iv] facilitates customising to construction specifications in line with the ongoing industrial digitalisation; and
[v] integrable with the existing post-processing software, such as Veta, to view and analyse the collected IC data.

In this paper, we provide the basic design concepts of the SPARC IC kit, its functionalities and capabilities with initial test results. The design concepts of the SPARC IC kit prototype will be further refined in the future based on the field trials undertaken on different materials using different roller types. The experiences gained through using the SPARC IC kit in actual construction projects will pave the way to develop robust and data-oriented specifications for IC to be used by the Australian road construction industry within the intelligent construction of pavement earthworks in line with the Industry 4.0 revolution.

2 CURRENT STATUS OF IC TECHNOLOGY IN AUSTRALIA

We undertook an online questionnaire survey to access the current state of the IC technology adaptation in Australia for both soil and asphalt compaction and understand the challenges in implementing IC in Australia from the end-user (contractors) point of view. Over 60 responses were received from the local road construction companies and agencies. Figure 2 summarises the survey results.

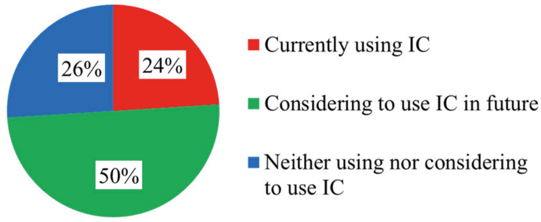

Figure 2. Current and future usage of IC in Australia

According to the responses recorded, IC is currently used for both soil and asphalt compaction equally, and it is expected that the proportion of IC usage for asphalt will show an increase in the future. Figure 3 shows the purpose of the usage of IC technology by the current users in Australia. The majority of the responses are either using IC technology or considering using it for both quality assurance (QA) and quality control (QC).

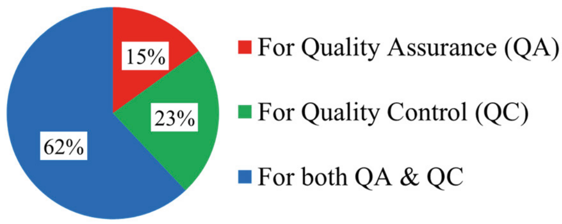

Figure 3. Purpose of the usage of IC technology

Based on the responses received for our online questionnaire survey, the followings are identified as issues associated with implementing IC technology in Australia that require further research:

- Responses indicated that IC technology is too expensive. A comprehensive cost-effectiveness study for IC in Australia is required to support the industry to decide whether it is worth investing in IC technology. In this regard, SPARC Hub IC projects undertake a comprehensive cost-effectiveness analysis of IC for both asphalt and soil.
- Responses indicated that the accuracy of ICMV is uncertain. A critical evaluation of the available ICMVs is required to identify the reasons for this uncertainty. Research activities should focus on developing methods to eliminate or reduce uncertainties.
- Responses indicated that lack of standards or specifications for the use of IC in Australia. A study should be undertaken to develop national wide IC specifications in Australia for asphalt and soil.
- The state-of-the-art IC technology is incapable of extracting the properties of each layer in a road pavement system (Sivagnanasuntharam et al., 2021). Instead, IC parameters recorded during compaction represent the properties of the equivalent pavement system, which is governed by the depth of influence of the roller.
- The industry is unsure about the post-construction usage of the IC data. IC data can be used to compare the performance of the road in the long run and can be used as

evidence to justify the compaction quality when any pre-mature failure occurs due to various reasons. IC data can also be used as a basis to make decisions for regular maintenance of the road sectors. In addition, IC data can be used as input for further developments such as the digital twin of road infrastructure.

- One of the challenges of IC is reported as 'setting up the system properly and ensuring the obtained data is accurate'. This issue is associated with retrofitting the IC kit to the existing conventional rollers. For example, if an accelerometer is supposed to be attached vertically to the roller but is attached at an inclined plane, measurements by the accelerometer may then have errors.
- The industry believes that IC technology can deliver uniform compaction. However, uniformity with respect to ICMV may not guarantee uniformity in the density due to the other factors that influence the measured ICMV. Therefore, a rigorous understanding of the correlation between ICMV and density is required for both asphalt and soil, considering other factors such as temperature for asphalt and moisture for the soil. In the SPARC IC projects, hypotheses are developed to improve the correlation between ICMV and density by taking into account the effects of these parameters. Experimental and numerical studies are undertaken to examine these hypotheses.

3 SPARC INTELLIGENT COMPACTION (IC) KIT

The SPARC IC kit is developed using Python language. The kit can run on both the Linux and Windows platforms. The prototype version is currently tested on Raspbian OS. The performance speed of the kit and raw data quality depends on the modular components of the hardware. The hardware of the kit is developed using state-of-the-art development circuit boards and components. The hardware of the kit is in modular form and the components can be easily replaced or upgraded. Figure 4 shows a picture of the 3D printed kit. It should be noted that for commercialisation purposes, these hardware components need to be integrated into one commercial-grade printed circuit board (PCB) and the enclosure of the kit needs to be designed to have at least an IP67 rating for outdoor use. Also, the codebase used for developing the analytics needs to be run through a software QA team to perform unit testing.

Figure 4. 3D printed SPARC IC kit

Figure 5 depicts a schematic overview of the functionality of the SPARC IC kit. The variables that are measured using sensors during the compaction process (i.e., the input parameters for calculations) are shown on the left side of the diagram and various intelligent compaction measurement values (ICMVs) determined by the kit are shown on the right side of the diagram.

The raw input data (as shown in Figure 5) for processing ICMVs are sampled from retrofitted sensors on the roller. The vertical and horizontal drum accelerations are measured with a triaxial accelerometer, roller speed and position are measured non-intrusively with high precision from a GPS-RTK unit, and the pavement material deformation data during each roller pass is measured by a high-precision laser array installed on the roller, respectively. Figure 6 schematically presents the hardware components of the kit and how they are integrated into a single-board computer (SBC).

The real-time raw sensor data is processed by the SPARC IC kit to produce different ICMVs and generate a colour-coded map. An example colour-coded map generated by the SPARC kit is shown in Figure 7. This map can be viewed by the roller operator in real-time during compaction and it can also be viewed remotely using a mobile phone or a computer. The kit records these ICMV data and can perform preliminary analysis without the need for additional equipment.

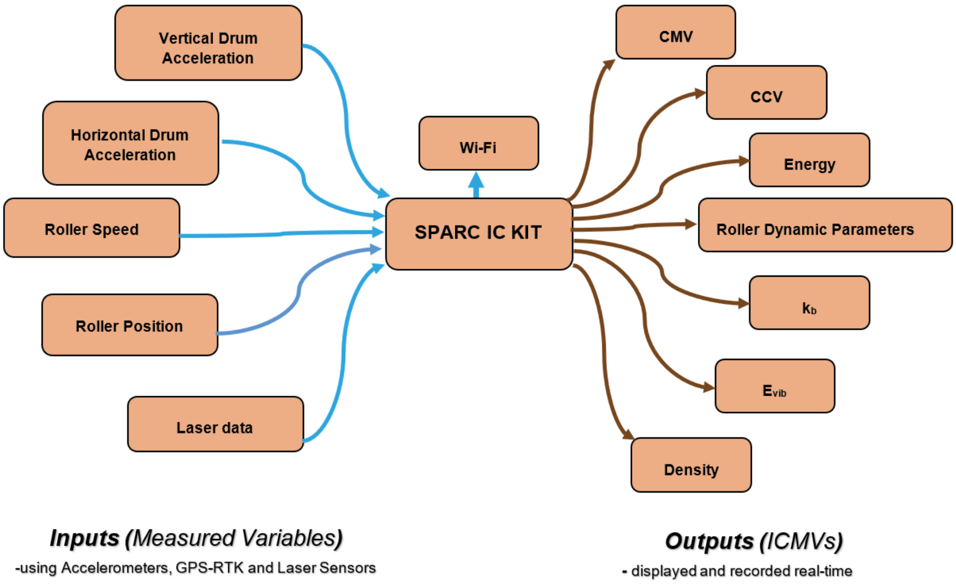

Figure 5. An overview of the functionality of the SPARC IC kit

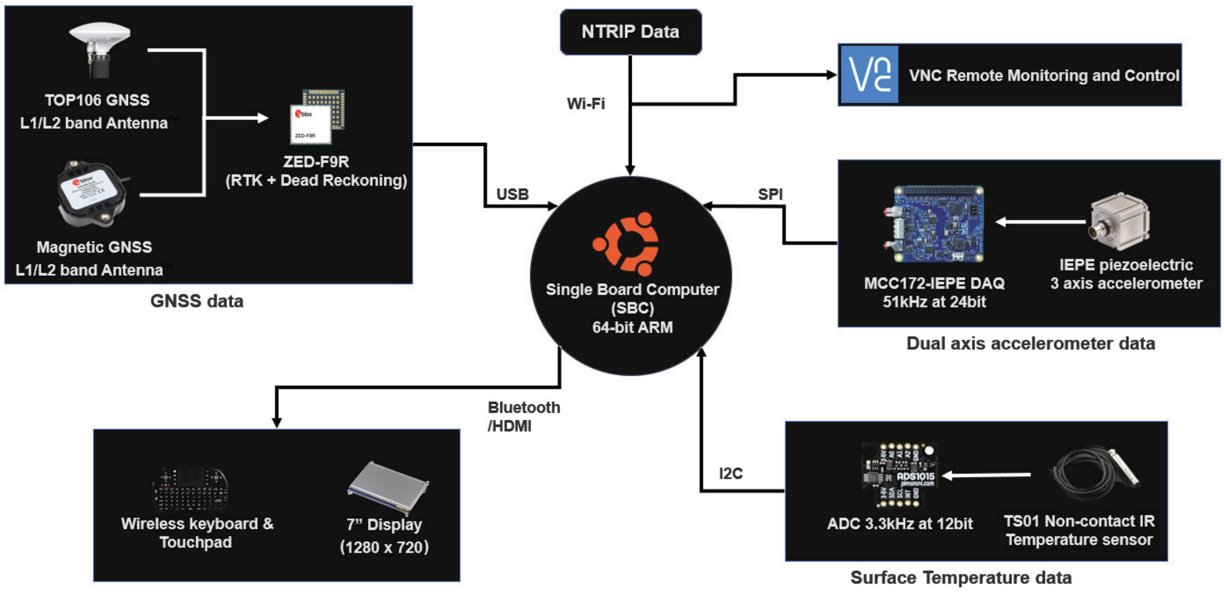

Figure 6. Hardware overview of the SPARC IC kit

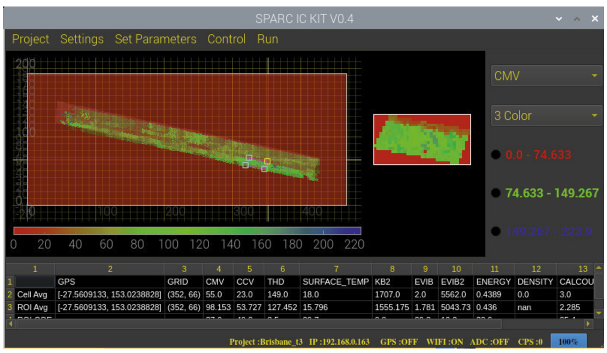

Figure 7. An example colour-coded map generated by SPARC IC kit for CMV parameter

4 INTELLIGENT COMPACTION METER VALUES (ICMVS)

Since the 1980s, various roller manufacturers and researchers have developed different types of measurement systems for assessing the ground condition based on the accelerometer readings, such as compaction meter value (CMV), compaction control value (CCV), roller integrated stiffness (k_b) and vibratory modulus (E_{VIB}). Intelligent compaction measurement value (ICMV) is the common term used for accelerometer-based vibration measurement systems mounted on vibratory rollers.

SPARC IC kit uses the raw data recorded by the accelerometer attached to the vibratory drum of the roller to calculate different ICMVs simultaneously in real time during compaction. Figure 8 illustrates the general procedure for calculating various ICMVs. As shown in this figure, the vertical displacement of the roller drum is calculated using the amplitude of the fundamental frequency (A_Ω) of roller vibration.

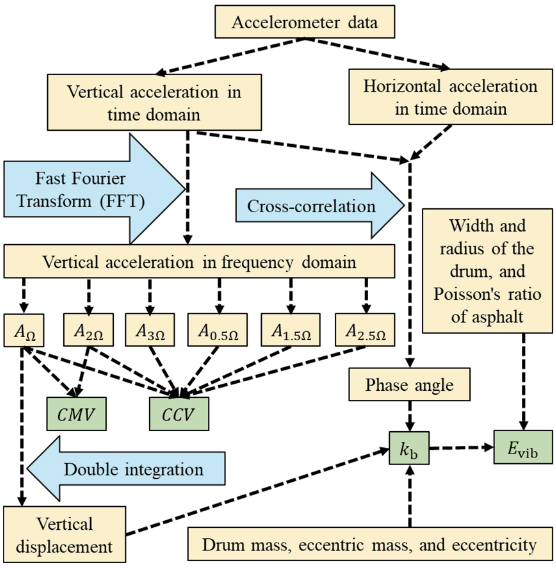

Figure 8. ICMV calculation procedure (Sivagnanasuntharam et al., 2022)

According to the roadmap for IC presented by the US Federal Highway Administration (2017), CMV and CCV are considered as Level 1 ICMV parameter while roller-integrated stiffness (k_b) and vibratory modulus (E_{VIB}) are considered as Level 3 ICMV parameter. The CMV and CCV require only the vertical acceleration of the roller drum as the input. During the compaction process, the accelerometer generates an acceleration signal in the time domain. Therefore, a fast Fourier transform (FFT) is performed to obtain the vertical acceleration signal in the frequency domain (Figure 9). In addition to vertical accelerometer data, the calculation of roller-integrated stiffness (k_b) and vibratory modulus (E_{VIB}) requires the following input parameters: drum width, drum radius, drum mass, eccentric mass, eccentricity, and Poisson's ratio of the material that can be pre-defined and the phase angle (defined as the phase lag between the excitation force of the roller drum and ground reaction force) that needs to be measured during compaction.

The phase angle can be measured with either a proximity sensor, such as the Hall effect sensor (Rinehart and Mooney, 2005) or an encoder. When an IC roller is manufactured by its original equipment manufacturer (OEM), this sensor is integrated inside the roller drum to measure phase angle (see Figure 1). However, retrofitting a conventional roller with such a sensor is not practicable. Accordingly, we have developed a novel method using the cross-correlation technique to estimate the phase angle without the need for a sensor. This method was successfully integrated into the SPARC IC kit, and it is being tested in the IC field trials.

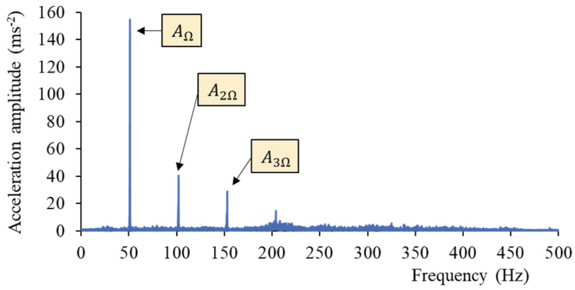

Figure 9. Vertical acceleration of roller drum in the frequency domain: an example (A_Ω, $A_{2\Omega}$, and $A_{3\Omega}$ are the amplitudes of the acceleration of the fundamental, first and second harmonic components)

5 CALIBRATION AND VALIDATION OF SPARC IC KIT

SPARC IC kit needs to be calibrated for all the ICMV parameters it calculates. The calibration process involves three main phases.

1. Primitive desk calibration was completed for both ICMV parameters and DAQ system to get the ICMV values in the desired range.
2. A 4-tonne double-drum roller was hired to undertake targeted field experiments in the SPARC test pit (5 m by 10 m). Controlled outdoor tests have been undertaken on unbound granular material and subgrade soil for the calibration of different ICMVs where the SPARC IC kit is attached to the roller to collect ICMV data processed in real-time (Figure 10). Figure 11 shows an example of ICMV data produced by the SPARC IC kit during the compaction of soil and unbound granular layers, respectively. The data collected using the kit is then analysed to improve the ICMV calculations. Inside the same test pit, tests have also been performed on the compacted soil using a Nuclear Density Gauge (NDG, soil density measurement), two types of lightweight deflectometer (LWD, soil dynamic modulus measurement), an L-band instrument (ELBARA III, proximal measurement of soil moisture) and a ground-penetrating radar (GPR, proximal measurement of soil moisture) to evaluate the potential use of these devices for enhancing the quality management of IC.
3. Targeted field trials are planned using an IC roller produced by OEM (original equipment manufacturer) to validate the ICMV parameters calculated by the SPARC IC kit. The OEM IC roller will be instrumented with the SPARC IC kit for this purpose.

Figure 10. A double-drum roller instrumented with SPARC IC kit (Site: SPARC test pit in Notting Hill, Victoria)

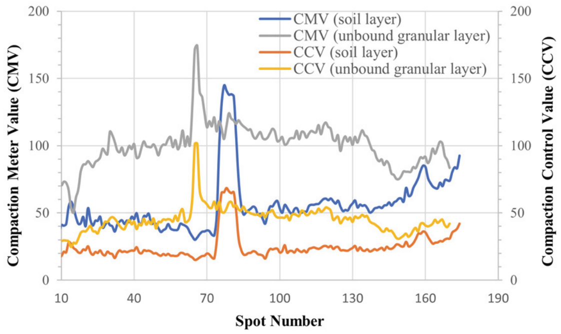

Figure 11. An example of CMV and CCV data generated by the SPARC IC kit during a roller pass

6 FUTURE WORKS

6.1 Post Processing Software

Post-processing software is being developed to process the raw accelerometer data, generate frequency spectrums, and produce ICMV values offline. This program helps analyse, validate and compare various performance parameters offline.

6.2 Proximal Measurement of Density

Based on our recent theoretical developments and experimental results, we have filed a patent on the innovation of proximal measurement of pavement material density and other properties in real-time during compaction. When this technology is fully developed, it has the potential to transform globally the geomaterials compaction to the highest level (Level 5 – a road map for IC by FHWA, 2017). The proximal measurement of density in real-time during compaction requires laser sensors, and the associated analytics can be integrated into the kit. The technique is currently being tested under different field conditions, and when it is fully developed, it will be integrated into the kit to display the estimated density of the material in a colour-code map along with ICMVs.

6.3 Calibration of Correction Factor for Underlying Support Effect

We identified that the effect of underlying support on ICMV (E_{VIB}) measured by IC rollers is one of the main causes of the poor correlation between ICMV and spot density measurements. We developed a practical method to decouple the influence of underlying support on E_{VIB}. The method was integrated into the SPARC IC kit to perform the real-time calculation of an index for the modulus of the material layer that is being compacted. The proposed method was tested in a small-scale asphalt testbed construction using an IC roller. However, it needs to be examined further for different materials and roller types.

6.4 Field Trials

The functionality of the SPARC IC kit is still to be tested comprehensively in field scenarios. This needs to be undertaken in phases of testing under different conditions and subsequent refinements. During these trials, performance requirements for the compaction material, such as target density and moisture content for soil, can also be incorporated for display and decision-making.

7 ACKNOWLEDGMENTS

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Powering Pile Design through Parametrics

E. J. Verrocchi[1], B.Sc, M.Eng and N. Ficatas[2], B.Com, M.Eng.

[1]Graduate Structural Engineer, ARUP, 699 Collins Street, Docklands, 3008, VIC, Australia; PH (+61) 9668 3349; email: Elise.Verrocchi@arup.com
[2]Structural Engineer, ARUP, 699 Collins Street, Docklands, 3008, VIC, Australia; PH (+61) 9668 6092; email: Nathan.Ficatas@arup.com

ABSTRACT

Parametric design skills are increasingly in demand within the engineering consulting industry due to ever-pressing time and budget constraints on medium- to large-scale infrastructure projects. The structural engineering team at Arup examined obstacles and manual processes encountered by the team during the design of a piled noise wall and endeavoured to automate, or at-least streamline, many of them. This resulted in a 94% saving in pile cross-sectional analysis computation time compared to manually inputting pile cross-section and design actions into analysis software and designing the pile using spreadsheets. The team's skills in parametric design in Grasshopper, a parametric modelling tool, and basic scripting were leveraged and developed throughout the process, providing an essential way of designing. The pile design process was selected to demonstrate the benefits of the parametric design due to it being a common design task on major infrastructure projects, such as pile wall design. The tool was developed with the intent of it being modular, employing a plug-and-play approach that connected the key stages of the design process with outputs, such as embodied carbon calculations and engineering drawings. McNeel Rhino, a 3D visualisation software, and Grasshopper were chosen for the tool development as they provided a user-friendly, visual programming interface and many plug-ins to existing automation workflows. Thus, enabling the engineer to interrogate the output of each stage of the design process both visually and numerically. The modular approach also enabled engineers to substitute ALP in place of Brom's calculation, as the design progresses.

Keywords: parametric design, parametric, geotechnical, pile, structural, automation

1 INTRODUCTION

The rationale supporting parametric design methodology has strengthened over time with growth in its adoption, further elucidating the time savings and efficiencies to be gained. Furthermore, as the design process is often iterative due to the dynamic nature of working with multiple stakeholders and collaborating with various engineering teams, the ability to quickly run a design from start to finish with updated inputs is highly beneficial.

This workflow was based on improving an existing workflow used in the design of a noise wall for a highway project in Victoria.

2 METHODOLOGY

2.1 Modularity

To enable flexibility in the use of this parametric tool across different design stages, it was important to identify the primary inputs to enable the tool to perform at each level. Naturally, this led to a modular design approach focussing on the inputs required for both high-level reference design and comprehensive detailed design. As a consequence of having different inputs for each stage, additional flexibility was required in the calculation components performing the engineering design process, hence reaffirming why modularity was so critical.

2.2 Inputs

To design the piles, the design actions at the top of the piles were needed, and this was subject to the ground conditions. The primary inputs required irrespective of the design stage were the wall alignment, topography, geotechnical design parameters, design loads, and initial structural properties and geometry of the piles and noise wall, as shown in Figure 1. For detailed design, additional inputs were required, such as additional and refined geotechnical parameters, inputs for wind loading and fatigue assessments.

2.3 Geotechnical pile design

The loading at the top of each pile was determined using the geometry of the piled wall itself and the specified wind pressure acting on the noise wall. A cantilever was idealised to simplify the problem with the intention being to emphasise the power of automation and allow subsequent users to further develop the tool. Subsequently, the design axial force, shear force, and bending moment under both ultimate limit state (ULS) and serviceability limit state (SLS) conditions were determined. From this, the required pile depth under ULS and SLS could be determined. These key design parameters were obtained using Brom's (1964) equations for lateral resistance of piles in both cohesive and cohesionless soils. Note this analysis considered a simplistic laterally loaded pile scenario and vertical effects have not been considered.

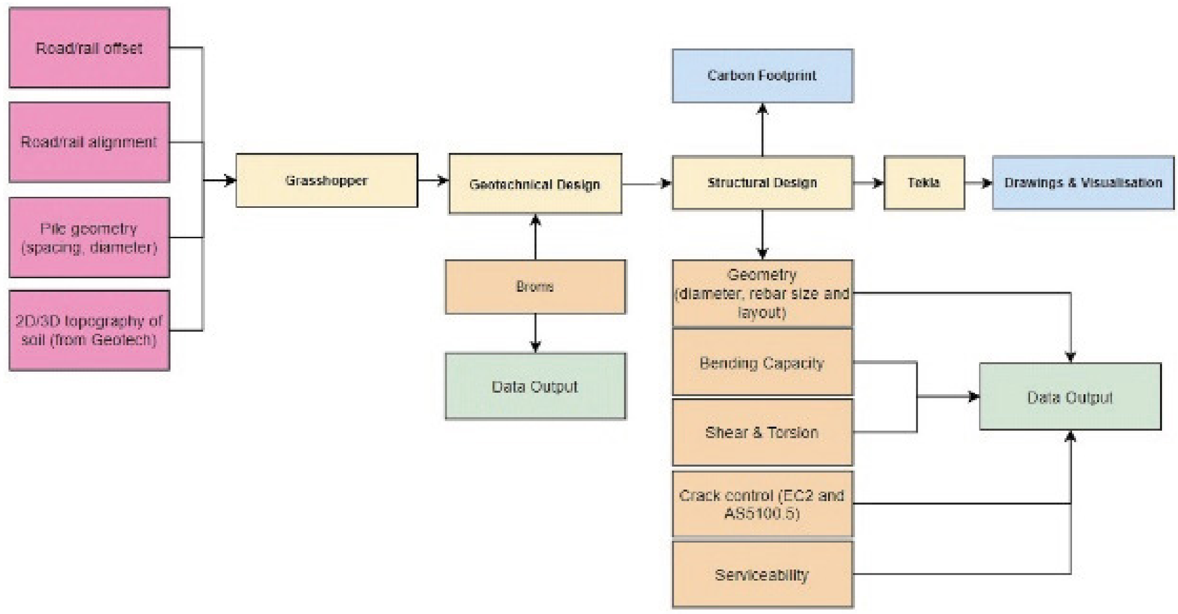

Figure 1. Modular workflow for pile design tool

Design parameters				
	Section geometry			
	Pile diameter, D =	1050	*mm*	
	Nominal cover, c_{nom} =	75	*mm*	*Refer Basis of Design report*
	Longitudinal reinforcement			
	Bar diameter, d_{st} =	40	*mm*	
	Number of bars, n_{st} =	20		
	Transverse reinforcement			
	Bar diameter, d_{sv} =	16	*mm*	
	Link spacing/pitch c/c, s =	150	*mm*	
	Material property			
	Capacity reduction factor, Φ=	0.7		*tbl. 2.3.2*
	Concrete characteristic strength, f'_c =	40	*MPa*	
	Concrete modulus of elasticity, E_c =	32800	*MPa*	*tbl. 3.1.2*
	Longitudinal steel yield strength, f_{sy} =	500	*MPa*	
	Transverse steel yield strength $f_{sy.f}$ =	500	*MPa*	
	Steel modulus of elasticity, E_s =	200000	*MPa*	
	Maximum nominal aggregate size, d_g =	10	*mm*	
	Design Loads			
	Shear force, V^*_{ULS} =	1000	*kN*	
	Bending moment, M^*_{ULS} =	200	*kNm*	
	Minimum bending moment, M^*_{ULS} =	789	*kNm*	*cl. 8.2.4.3(a)*
	Axial load, N^*_{ULS} =	-100	*kN*	*$-_{ve}$ compression*

Figure 2. Original spreadsheet for design shear strength of circular reinforced sections

When a project progresses to detailed design, Oasys ALP, a lateral pile design software, can be substituted into the Grasshopper workflow replacing Brom's calculations to determine shear forces and bending moments induced along each pile when subject to lateral loads and bending actions at top of pile.

2.4 Structural pile design

Based on the key design actions, the pile reinforcement required to withstand the design actions was determined according to AS 5100.5:2017. An established spreadsheet for pile shear capacity design, shown in Figure 2, was converted into a more practical digital tool.

We leveraged a framework developed in-house, known as ArupCompute, to script the calculations rather than Excel. One method of querying this tool was through a web interface shown in Figure 3. Coding this tool presented several key benefits:

- All calculations were verified once coded
- Any change to the coding for the calculation would alert the user/engineer
- Reduced the chance of human error in dragging incorrect cells, modifying formulae in Excel, or using unverified design spreadsheets

The pile section analysis was also conducted parametrically by drawing on the work of colleagues in Germany who developed a Grasshopper plug-in for Oasys AdSec. AdSec works by defining a reinforced concrete cross-section, applying relevant load cases, and analysing the pile under both ULS and SLS conditions to primarily determine ULS utilisation, Figure 4, and SLS crack widths.

Case	Load			ULS Status	Utilisations	
	F_x (kN)	M_{yy} (kNm)	M_{zz} (kNm)		Load	Deformation
1	-4400	150	0	✓	65.9%	36.24%
2	-220	220	0	✓	88.62%	54%

Figure 4. AdSec ULS results for two critical load cases

Any changes to the pile geometry would immediately update the analysis inputs and subsequent design process. This was controlled by the engineer to allow AdSec to run only when required, rather than after every update. AdSec was then used in combination with PowerBI to identify which piles were most highly utilised, drastically reducing computation times, as summarized in Table 1.

*Table 1: Timed run for analysis of twenty-five piles in AdSec**

Method	Task (test run 25 piles)	Time
Traditional method	Create individual AdSec files for each pile and manually apply loads	~1300s
Parametric method	Run each pile through AdSec (full creation of section per pile, apply loads, analyse)	~80s
	Create list of design actions for all cases and piles, run through single AdSec section**	~4.5s
	Run single max load case through single AdSec section**	~2.5s

*Assumes pile cross-sections are identical for all twenty-five piles
**Only one section initially loaded into AdSec, and design actions updated

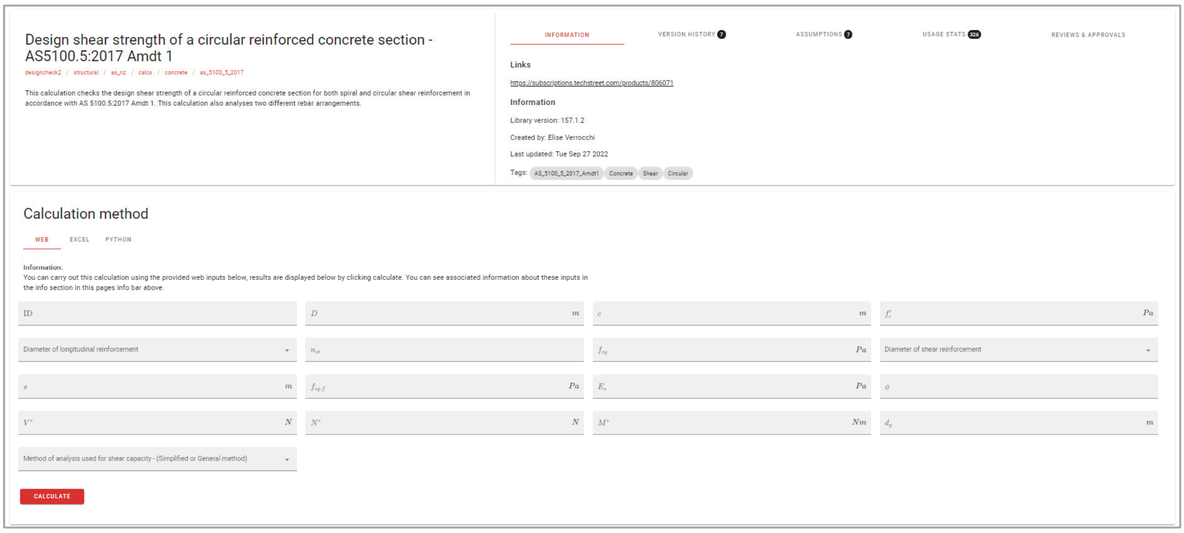

Figure 3. ArupCompute interface for design shear strength of circular reinforced sections

2.5 Design rationalisation

Structural pile design for a noise wall is often rationalised. This means, instead of each pile being designed individually based on its design actions and utilisation, piles are grouped based on the magnitude of their design actions to keep piles as close to 100% utilisation as possible. Therefore, instead of having one pile cross-section for the entire wall, you may have three to four different cross-sections implemented based on how highly utilised each pile at each location is.

Unlike a traditional design method, this workflow provides a design benefit due to the speed in which analysis can be carried out. Hence, each cross-section can be analysed to calculate the utilisation for each permutation. Consequently, the most optimum of the cross-sections can then be assigned to each pile.

2.6 Data visualisation

Parametric design enables the design of each structural element within the pile wall, therefore data visualisation of key metrics such as pile length, pile utilisation, total material quantities, and embodied carbon is possible. This was carried out using PowerBI, a data visualisation tool to display minimum required pile length, pile utilisations, crack width, and embodied carbon (Figure 5).

2.7 Drawing automation

With the analysis performed and design carried out, the next step in the overall workflow was to link the design work with the drawing automation process. Key parameters, including pile diameter, longitudinal reinforcement diameter, and number of longitudinal reinforcing bars, was uploaded to a database. Arup technicians then extracted and subsequently ran this data through their parametric workflow to generate 3D models and associated engineering drawings. The power of this process is that for any change in the geometry or loading during the analysis and design stages, the design can be re-run with minimal manual steps and ultimately the resultant information can be reflected in the models and engineering drawings within only a few minutes. Meaning it took only a few minutes to re-generate a set of ready-to-print engineering drawings, offering up to a 90% time saving. This further highlighted how parametric workflows can considerably speed up the drawing production process, particularly when there are changes to the design.

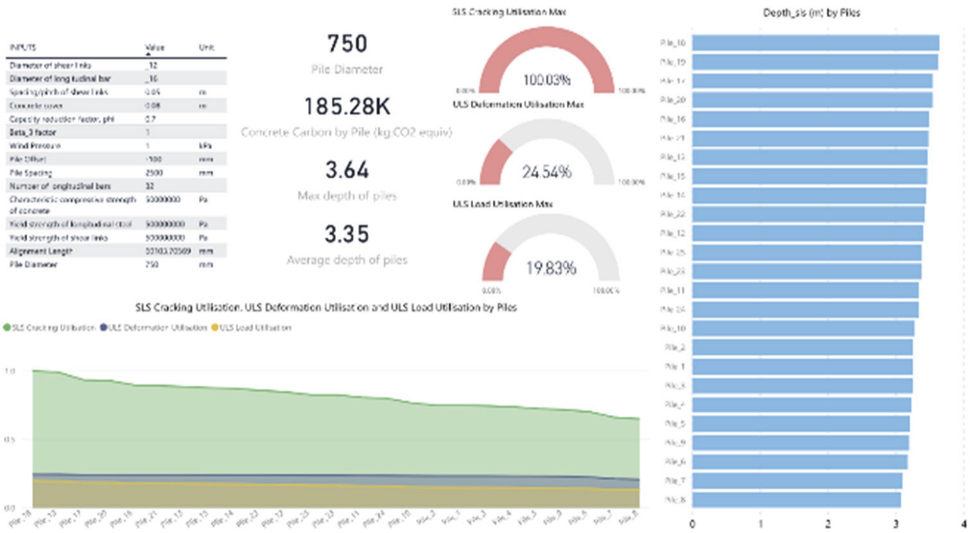

Figure 5. PowerBI data viewing dashboard produced for one noise wall design

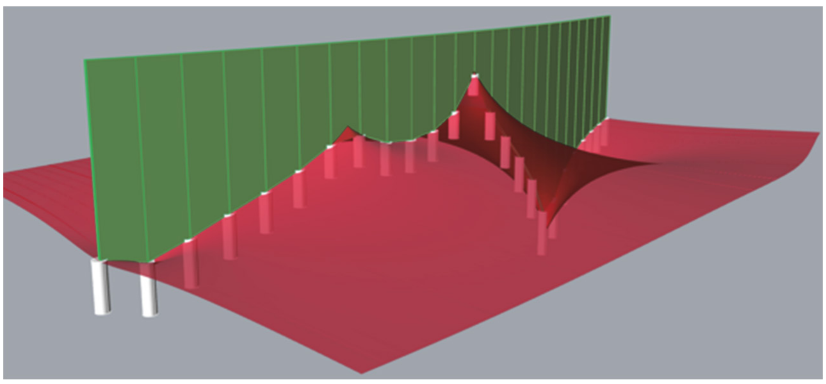

Figure 6. Real-time Rhino visualisation of topography and pile length with the topography exaggerated for display purposes

2.8 Ability to interrogate the tool

A fundamental consideration during tool development was ensuring that the tool did not resemble a "black box" and was instead completely interrogable. This was achieved through the focus on modularity, allowing the outputs to be interrogated at each step, in addition to using ArupCompute and its auto-generation of complete calculation sets to alleviate engineers' concerns surrounding providing inputs to an un-interrogable tool.

Not only was the tool easy to interrogate numerically, the noise wall layout and pile length were also displayed visually and updated in real-time in Rhino, as shown in Figure 6. This allowed the engineer to see the topography of the ground in which the piles were being constructed and the variation in pile length under different ground conditions.

2.9 Embodied carbon

Sustainability has become a key metric on projects and is calculated primarily through total embodied carbon. Hence, it was essential to include carbon calculations within the script to enable comparisons between different design methodologies. The Environmental Performance in Construction (EPiC) database (2019) was used to calculate total embodied carbon for cradle-to-gate (stages A1-A3) analysis of the piles.

3 LESSONS LEARNT

There were important lessons learnt from the process of developing a parametric pile design workflow. One of the major challenges encountered during the development of the tool was the magnitude of the task at hand, including branching out of our structural engineering domain to engage and understand geotechnical inputs and requirements. Specifically, managing the data output from ALP and modifying it such that it could be fed into the Grasshopper tool.

4 NEXT DEVELOPMENTAL STEPS

As with any automation tool, there are always additional features that can be added to refine and expand the current workflow. With continual global development of the Arup AdSec API, this will expand the functionality of the section analysis in Grasshopper whilst accelerating the process of running through larger datasets. Furthermore, with more sophisticated geotechnical plug-ins for Grasshopper in the pipeline, more accurate geotechnical analysis can be carried out as part of the workflow to further refine the design process.

5 NEXT DEVELOPMENTAL STEPS

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6 CONCLUSION

This case study proved how powerful and essential parametric workflows can be within engineering design. By identifying manual processes within our traditional workflows to automate, computational times can be dramatically reduced, up to 94% time saving in pile cross-sectional analysis computation time compared to traditional methods. This allows for more time for engineers to interrogate the inputs and outputs of each stage of the design process whilst further optimising the design presented to the client. In turn, savings in time and budget can be realised in addition to reducing the embodied carbon for cradle-to-gate of noise wall design.

7 REFERENCES

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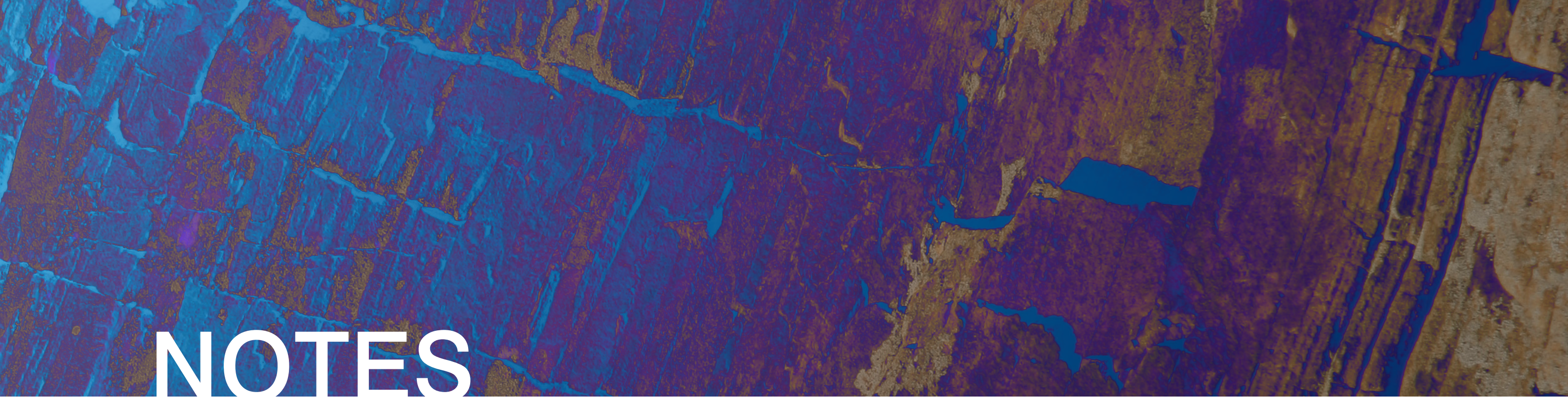

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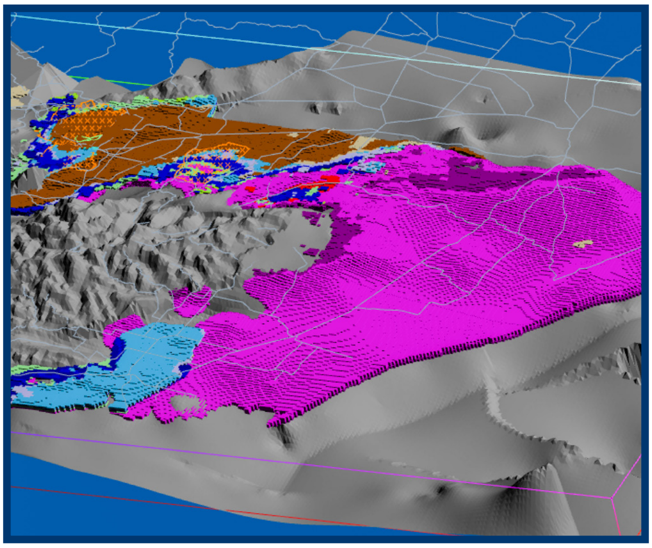

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