

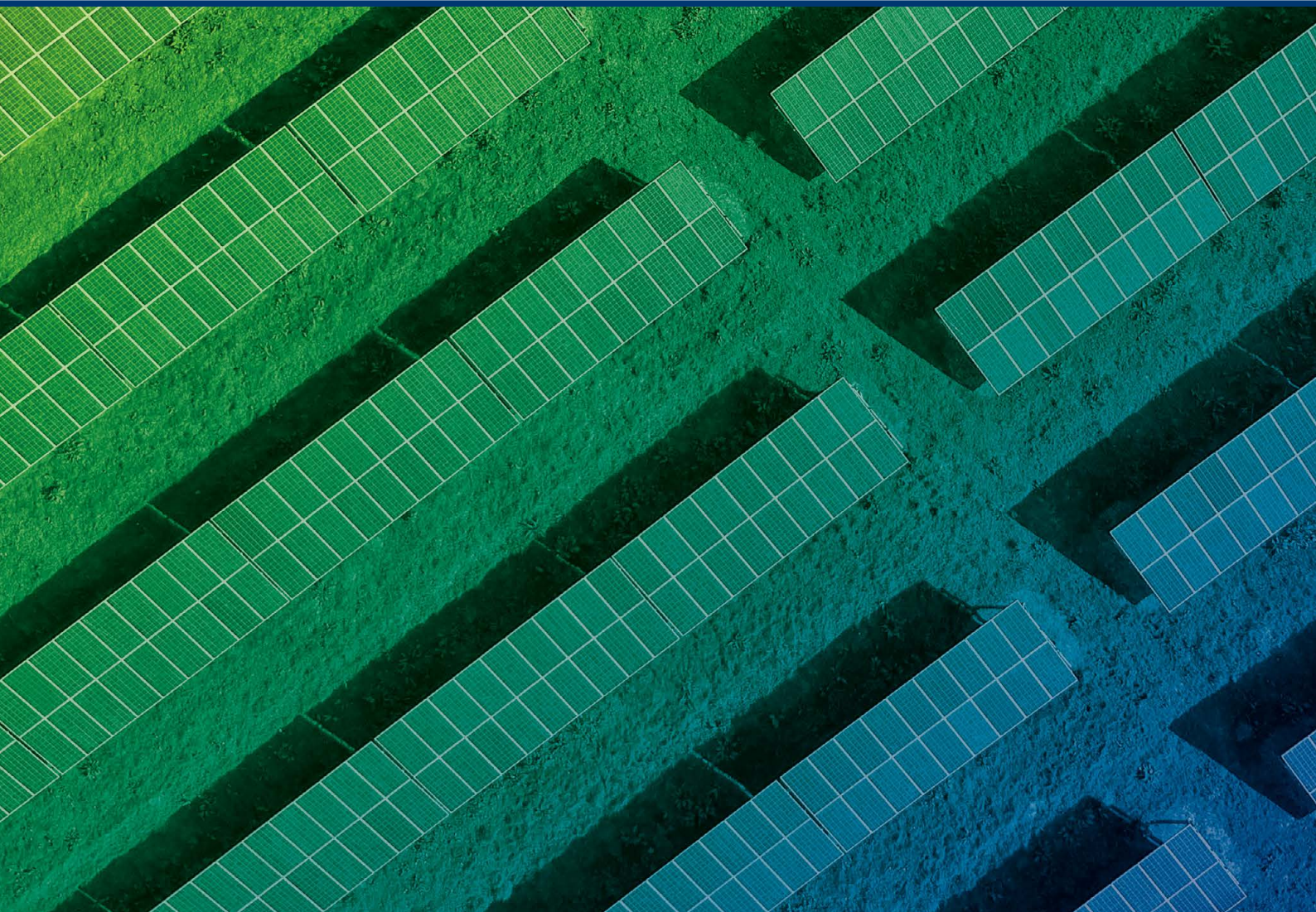


PROCEEDINGS

2020 AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIAN SYMPOSIUM

Sustainable Geotechnics – Excellence in Planning, Design and Construction

Wednesday, 18 November 2020, 8:25am – 4:20pm
An online-only event



AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER



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PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2020 Australian Geomechanics Society Victoria Symposium held on 18 November 2020. As this was a year heavily disrupted by COVID-19 and due to the associated restrictions on public gatherings, the symposium was an online-only event.

At the start of a new decade, our society faces many challenges of which engineers and geotechnical professionals are at the forefront. These challenges include managing our limited resources, finding new sources of energy generation, minimising waste, managing contaminated sites, and the risks and hazards associated with a changing climate. Addressing these challenges requires the innovative use of optimised and lean investigation, design and construction methods, use of alternative or recycled products, and finding ways to “do more with less”.

The Symposium brought together professional engineers, geologists, geophysicists, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of sustainable geotechnics, and provided attendees with an exciting opportunity to help define what sustainability means for our profession. Best practices, case histories and innovative solutions for dealing with these challenges were presented and discussed.

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ABOUT THE KEYNOTE SPEAKERS

ROSS ROBERTS

Auckland Council

Ross is an engineering geologist with a keen interest in geohazards and sustainable development. He is currently Chair of the New Zealand Geotechnical Society as well as Auckland Council's Geotechnical and Geological Practice Lead which involves managing the geotechnical risk across the Auckland metropolitan area. He has worked for consultants and contractors on projects including major highway construction, railway asset management, water pipeline scheme assessments, landslide assessment and remediation, microtunneling and bridge foundation design. His current focus is on improving the resilience of development in New Zealand while minimising the impacts.



PROFESSOR ARUL ARULRAJAH

Swinburne University

Professor Arul Arulrajah leads the Geotechnical Engineering group at Swinburne University of Technology. Arul has been involved as a Chief Investigator in 28 research projects totalling AUD\$11.5 Million which includes 4 ARC Linkage grants, 1 ARC Training Centre, 3 ARC LIEF, 15 competitive state government grants and 6 industry grants. He is the first-named investigator and project leader in 19 projects totalling AUD\$2.6 Million. He is involved in geotechnical engineering research topics such as recycled materials, ground improvement, pavement geotechnics, in-situ testing, field instrumentation, land reclamation, dredging and deep compaction works. He has supervised 12 PhD students to completion in this area of geotechnical and sustainability research. He is presently the Deputy Training Centre Director and Technical Program Coordinator for the ARC Training Centre for Advanced Technologies in Rail Track Infrastructure. Professor Arulrajah is the author of 2 books, 5 book chapters, 290 journal publications and 93 conference publications to date. His research has been recognised by international and Australian awards. This includes the Telford Premium Award from the Institution of Civil Engineers, UK (2009), Australian Pavement and Recycling Association Highly Commended Awards for Excellence in Sustainability and Research (2018 and 2012) and the Shamsheer Prakash Prize for Excellence in the Practice of Geotechnical Engineering (2010). He is also the recipient of 4 Swinburne Vice Chancellor Awards and 2 Swinburne Researcher Awards for his research activities. He has extensive experience in the geotechnical engineering design, site implementation and project management of numerous large civil engineering infrastructure projects in Australia, Singapore and Malaysia, based on his 14 years of industry experience prior to joining Swinburne University in 2006.



PATRICK WONG

PKW Geosolutions

Patrick has been a consulting geotechnical engineer since graduating from The University of Sydney with First Class Honours in Civil Engineering in 1978. Patrick joined the Coffey Group in 1979, and has held various leadership positions within Coffey including directorship of the operating company during the period 1991 to 1993, and Manager of Eastern Region during 1995 to 2000. Patrick has a wide range of geotechnical design expertise, including deep basements, deep foundations, soft ground engineering and ground improvement. As well as all states of Australia, Patrick has conducted major projects in countries such as South Korea, Singapore, Malaysia, Vietnam, United Kingdom, Dubai, Jordan and Egypt. Patrick currently runs his own consultancy PKW Geosolutions Pty Ltd and is a Senior Consultant at Coffey Services Australia Pty Ltd where he continues to enjoy the role of technical leadership on major projects and mentoring of staff.



TIMETABLE

8:25 am Welcome and opening remarks

SESSION 1 CARBON EMISSIONS

8:40 AM KEYNOTE ADDRESS

Climate change, sustainable development and geotechnical engineering: A New Zealand framework for improvement
Ross Roberts, Auckland Council

9:20 am Presentation

Comparing the embodied carbon and local environmental impact of common geotechnical foundation solutions for the Australian market
Jared Fetherston, Keller Australia

9:35 am Presentation

Energy driven piles in Australia: Design and construction lessons from a trial at Fishermans Bend
Chris Price, Wagstaff Piling and Yu (Cherry) Zhong, University of Melbourne

9:50 am Sponsor presentation

Chadwick Geotechnics
Tim Chadwick

10:00 am Questions

10:10 am Morning tea

SESSION 2 SUSTAINABILITY AND THE RE-USE OF FORMER QUARRIES

11:00 AM KEYNOTE ADDRESS

Working Towards Net Zero Emissions – Role of Geo-professionals
Patrick Wong, PKW Geosolutions

11:40 am Presentation

A settlement hazard risk management framework for the development of backfilled open-cut quarries
Manuel Neves, Tonkin & Taylor

11:55 am Presentation

Backfilled Quarry Development with Inbuilt Landfill Gas Solution
Claire Howell, Senversa

12:10 pm Presentation

Brown gold: redevelopment of former quarry and landfill sites
Stuart Colls, Golder

12:25 pm Questions

12:35 pm Lunch

SESSION 3 RECYCLED MATERIALS

1:30 PM KEYNOTE ADDRESS

Recent advances in the usage of recycled materials in transportation geotechnics
Arul Arulrajah, Swinburne University

2:10 pm Presentation

Blends of recycled materials as sustainable alternatives for backfilling sewer trenches
Ehsan Yaghoubi, Victoria University

2:25 pm Sponsor presentation

Global Synthetics
Amir Shahkolahi

2:35 pm Questions

2:45 pm Afternoon tea

SESSION 4 TECHNOLOGY BASED SOLUTIONS

3:00 pm Presentation

Innovative Geotechnical Design, Smart Construction and Optic Fibre Sensing
Mahdi Distfani, University of Melbourne

3:15 pm Presentation

An Innovative Approach for the Installation of Vibrating Wire Piezometers
Mark Chapman, Insitu Geotech Services

3:30 pm Presentation

Sustainable Engineering Solution for Slope Stability with Anchor Reinforced Vegetation System (ARVS)
Amir Shahkolahi, Global Synthetics

3:45 pm Presentation

Sustainable Lining for Underground Hard Rock Openings
Keith Kong, Coffey

4:00 pm Questions

4:10 pm Closing address

4:20 pm Finish

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SESSION 1
CARBON
EMISSIONS

Keynote Address

Climate change, sustainable development and geotechnical engineering: A New Zealand framework for improvement

R. C. Roberts¹, CGeol, CMENZ, PEngGeol

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ABSTRACT

Climatic warming caused by the emissions of anthropogenic greenhouse gases is occurring across the globe. These changes will increase the exposure of the built environment to hazards such as sea level rise and coastal inundation and exacerbate existing hazards such as expansive soils and landslides. In New Zealand, the built environment and its construction is responsible for about 20% of the emissions that are the primary cause of climate change. The way our built environment is designed must change to adapt to these future increases in hazard and must also mitigate emissions where possible to limit future increases in hazard to manageable levels. This paper describes climate change effects where they have overlaps with geotechnical design and hazard assessment (with particular reference to Auckland as an example), discusses the impact that these changes are expected to have on geotechnical practice in the coming years and decades, and presents a framework for managing these in the design processes.

Keywords: climate change, geotechnical engineering, sustainable development, adaptable design

1 INTRODUCTION

1.1 Climate change

Since the last ice age, which ended approximately 12,000 years ago, Earth's climate has been relatively stable. This stability in temperature and sea level was crucial for the development of modern civilisation. However, since about 1900 average temperatures around the globe have been increasing. This warming trend is expected to have a drastic impact on modern life which is tailored to the stable climate in which it developed.

Increasing temperatures have been strongly linked to emissions of anthropogenic greenhouse gases (Figure 1), such as carbon dioxide (CO₂). Since the 1950s many of the observed changes are unprecedented, including environmental changes such as increased atmospheric and oceanic

temperatures, decreased mass volume of snow and ice worldwide, and a significant rise in global mean sea level (IPCC 2014a).

The decisions we make today need to consider the changing environment and the potential disruptions it will cause. A typical structure constructed today will still be in use in 2070 and beyond. However, the climate it encounters will be significantly different. Planning decisions and investment in major infrastructure made today will have consequences over even longer timeframes (for example, once a town is built its existence is near-permanent).

This paper discusses the impact that these climatic changes are expected to have on geotechnical practice in the coming years and decades and presents a framework for managing these in the design process.

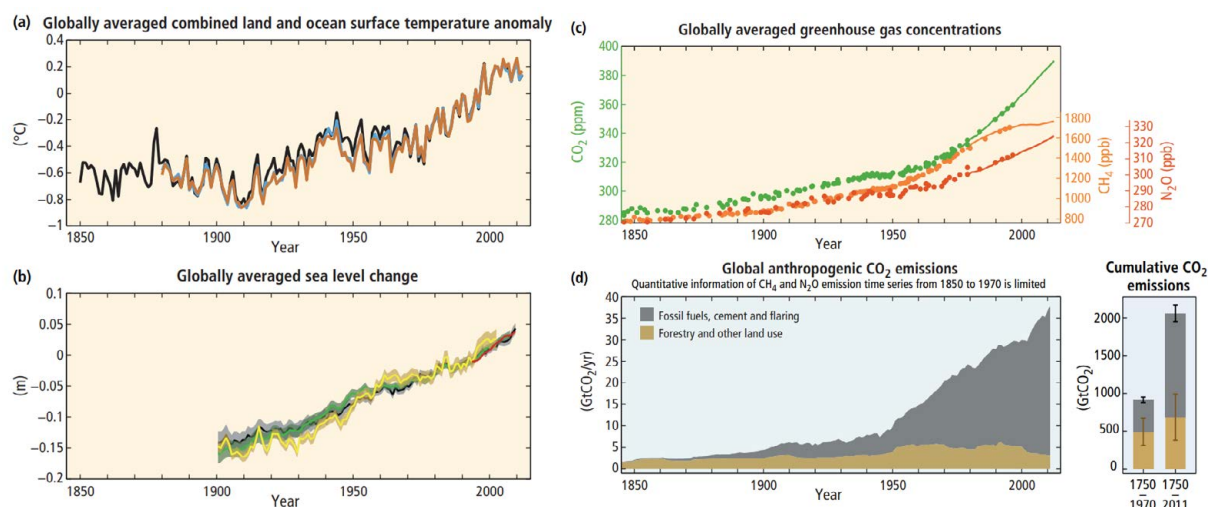


Figure 1. The relationship between observations (panels a, b and c with yellow background) and emissions (panel d, light blue background). After IPCC 2014a Figure SPM.1.

1.2 Sustainable development

Sustainable development is development that meets the needs of the present without compromising the capacity of future generations to meet their own needs (United Nations World Commission on Environment and Development, 1987 “Brundtland Report”). It is the broader framework under which the management of climate change’s impact on the built environment falls.

Published in 1987, the Brundtland Report clearly highlighted the threat posed by climate change:

“Little time is available for corrective action. In some cases we may already be close to transgressing critical thresholds. While scientists continue to research and debate causes and effects, in many cases we already know enough to warrant action. This is true locally and regionally in the cases of such threats as desertification, deforestation, toxic wastes, and acidification; it is true globally for such threats as climate change, ozone depletion, and species loss. The risks increase faster than do our abilities to manage them.”

Seventeen UN Sustainability Goals were adopted by all UN Member States in 2015 as part of the 2030 Agenda for Sustainable Development, which set out a 15-year plan to achieve the goals (United Nations, n.d.). Goal 13 requires member states to “Take urgent action to combat climate change and its impacts”, and includes the following targets:

- (i) 13.1 Strengthen resilience and adaptive capacity to climate-related hazards and natural disasters in all countries.
- (ii) 13.2 Integrate climate change measures into national policies, strategies and planning.

Partly because of this, there is a significant effort in member countries to achieve these targets in the coming decade.

2 CLIMATE CHANGE IMPACTS

2.1 General climate change impacts

Goal 15 of the UN Sustainability Goals focuses on land degradation, the loss of life-supporting land resource due to processes such as soil erosion, desertification, deforestation, and acidification.

Land degradation is exacerbated by climate change which can cause increases in:

- Rainfall intensity and changing rainfall patterns.
- Flooding frequency and severity.
- Drought frequency and severity.
- Heat stress.
- Wind strength.
- Coastal erosion and more frequent and extensive inundation caused by sea-level rise and wave action.
- Permafrost thaw.

Land degradation and increasing instability adds significant land use pressure in regions where land suitable for building was already scarce (IPCC, 2019).

2.2 Geotechnical impacts

Land degradation and changes to land stability have a significant impact on the built environment. Climate change impacts of particular relevance to geotechnical engineering in New Zealand include drought and its impact on expansive and settlement-prone soils, coastal erosion and its impact on coastal cliffs, and rainfall intensity and its impact on land stability.

These issues create financial and social problems as well as engineering problems. Properties in hazard zones may struggle to get insurance, exposing the owners to significant loss. Even if the building platform can be protected, the loss of adjacent land can significantly devalue a building.

2.2.1 Drought

Forecasts of future total precipitation rates vary geographically with some areas expecting an increase, and others a reduction. In Auckland, total rainfall is expected to decrease only slightly. However, changes in seasonal patterns may result in wetter autumns and drier springs (Figure 2). An increase in the number and severity of droughts is expected to result in a greater range of groundwater levels, with the seasonal changes expected to result in lower groundwater levels in summer. This effect will be exacerbated by higher rates of evapotranspiration and in some areas because of increasing groundwater abstraction for human use.

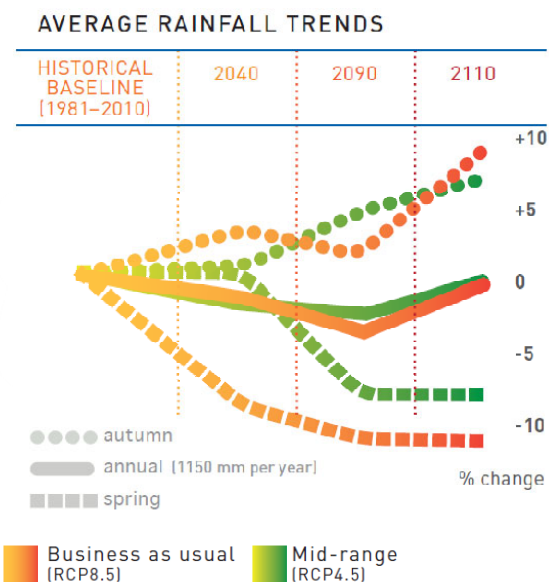


Figure 2. Average rainfall trends for Auckland to 2110 (Lorrey et al, 2018). RCP8.5 represents a scenario where emissions continue to rise at current rates, while RCP4.5 represents an emissions peak at 2040, followed by a slow decline likely to result in global temperatures rising 2°C by 2100.

These more extreme summer lows are expected to increase shrinkage of expansive soils and increase (often irreversibly) settlement in normally consolidated clays and peat. This was highlighted by the Auckland's dry summer of 2020. Between 1 January and 21 May just 126 mm of rain was recorded at NIWA's Mangere weather station - an amount less than two-thirds of normal - making the drought one of the worst since the early 1900s, and resulting in a doubling of reports of building damage (NZ Herald, 2020).

Slope stability is also likely to be affected by drought. Although (as noted below) drier conditions are often advantageous for slope stability, some drought effects can destabilise slopes, particularly:

- Loss of vegetation
- Desiccation cracking
- Reduction in soil suction

Extended droughts will kill some vegetation, particularly species that are adapted to a specific climatic zone. Over time, more drought-tolerant species will likely take over, but in the interim the loss of the stability and erosion control provided by root reinforcement and foliage cover may be significant.

Desiccation cracking due to soil drying is largely governed by the soil's plasticity, the temperature, and the number of volume change cycles experienced. These cracks form natural weaknesses and enhance vertical permeability allowing rainstorms to rapidly saturate specific horizons, particularly along existing weak planes that are common at weathering or lithological boundaries.

In partially saturated soils, reduction in soil suction causes a corresponding reduction in shear strength. This is largely governed by the pore size with fine grained soils retaining strength noticeably longer than soils with larger particles at similar saturation levels (Vahedifard et al, 2018). This loss of strength is challenging to predict and could result in unforeseen failures.

2.2.2 Coastal erosion and sea level rise

Sea levels around Auckland are forecast to increase by between 0.4 m and 1.1 m by 2100 (Figure 3).

An increase in coastal inundation and erosion rates is predicted for most areas. Cliff toe erosion will increase slope instability risks, and geotechnical structures will be more exposed to flooding and erosion.

As high-value land is eroded, demand for coastal protection measures is likely to increase. It is already common to install palisade walls around cliff-top properties to protect them against future erosion, and demand for sea walls is increasing.

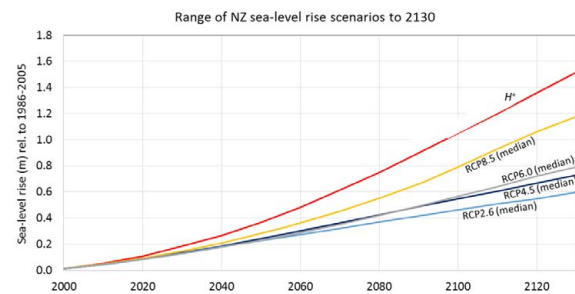


Figure 3. Range of sea level rise scenarios for NZ to 2130 (Stephens, 2017)

2.2.3 Rainfall intensity and storminess

Across New Zealand, many areas are forecast to receive more intense rainfall, including Auckland (Figure 4) where extreme rainfall intensity is forecast to increase by 15-30% by 2090 (Lorrey et al, 2018).

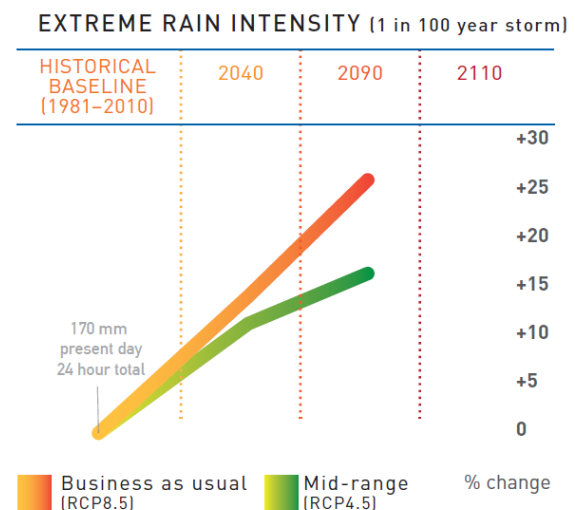


Figure 4. Extreme rain intensity scenarios for Auckland to 2090 (Lorrey et al, 2018).

In areas already forecast to receive an increase in total precipitation, higher groundwater levels and pore water pressures are expected and will contribute to a reduction of shear strength, a reduction in soil suction and cohesion, and an increase in the weight (wet density) of the slope materials, all decreasing slope stability. Therefore, less rain will be required to fall in each event to reach a critical level that can cause a slope to fail. With an increase in the rainfall intensity this is likely to cause a significant increase in landslide susceptibility.

Conversely, a reduction in total precipitation would normally result in more stable conditions. However, the forecast increase in rainfall intensity may cause higher infiltration into dry, cracked ground during storm events (where the soil and bedrock allow it) and an increase in subsurface drainage and throughflow. Together these contribute to the build-up of perched water tables, the reduction of effective normal stresses and the shear strength, again contributing to slope instability (Gariano and Guzzetti, 2016).

Increased rainfall intensity will also increase surface runoff (overland flow) and the related surface erosion processes. These may in turn facilitate debris flow initiation and enlargement, as well as contributing to increased erosion of river banks, increasing bank instability. The instability of the river banks may propagate upslope or laterally, initiating new landslides or reactivating old, dormant ones.

Increases in the number and severity of storms is also forecast, which has implications for wind and wave loadings on structures, as well as increasing rates of coastal erosion. Stronger winds are likely to result in further destabilisation of slopes as instability can be triggered by trees blown over in storms.

2.2.4 Coastal groundwater level rise

As sea levels rise, groundwater in the vicinity of the coast is also likely to rise. In areas where the groundwater is already shallow, such as low-lying alluvial plains, flooding will likely be exacerbated by reduced stormwater infiltration. Shallower water will affect the strength of soils beneath foundations, potentially causing widespread structural damage, and more land will become susceptible to liquefaction during seismic shaking (Quilter et al, 2015).

2.2.5 Multi-hazards

Climate change is predicted to increase the risk posed by individual extreme events. However, multi-hazard scenarios where the occurrence of simultaneous or sequential events is considered to result in a much larger total risk.

For example, a period of intense drought (causing desiccation cracking and vegetation die-back) followed by an intense rainfall event (causing erosion of now unprotected surfaces, and rapid saturation of deep soils through desiccation cracks) would likely lead to much higher risk from landslides than either of these two events alone.

3 CLIMATE CHANGE STRATEGIES

In general responses to climate change responses are split into two categories: mitigation and adaptation.

Mitigation focuses on our ability to control emissions and is a human intervention to reduce the sources or enhance the sinks of greenhouse gases (IPCC, 2014b). The goal of mitigation is to stabilise greenhouse gas levels in a timeframe sufficient to:

- Allow ecosystems to adapt naturally to climate change.
- Ensure that food production is not threatened.
- Enable economic development to proceed in a sustainable manner.

Adaptation is the process of adjustment to actual or expected climate changes and their impacts. In human systems, adaptation seeks to moderate or

avoid harm or exploit beneficial opportunities (IPCC 2014c).

Because of the quantity of greenhouse gases added to the atmosphere in the past few centuries, we are already committed to some level of climate change and therefore to some level of adaptation. Without reducing our current rate of emissions, these changes will become much more severe. Therefore, it is generally accepted that both approaches are required in parallel. Adaptation is essential to respond to the changes that will happen, and mitigation is essential to limit these changes to manageable levels.

Adaptation and mitigation each have the potential to contribute to, or to impede, sustainable development. The best outcomes will occur when adaptation and mitigation work together to reduce risks of disruptions from climate change. Trade-offs between adaptation and mitigation and between economic goals and environmental goals may be required. In some cases, adaptation may increase greenhouse gas emissions (e.g. increased air conditioning in response to higher temperatures). In other cases, mitigation may impede adaptation (e.g. emissions trading may increase the cost of concrete, making it unaffordable for some countries to invest in relocating critical infrastructure away from hazard zones).

Climate change requires new approaches to sustainable development that consider the complex interactions between climatic, social and ecological systems. Climate-resilient pathways need to be defined that show how development can take place over time in a way that combines adaptation and mitigation to realise the goal of sustainable development. These pathways are iterative, continually evolving processes for managing change within complex systems (Denton et. al., 2014).

These approaches have secondary benefits. Strategies for climate change responses and strategies for sustainable development are often highly interactive, and in some cases, reducing the risk related to climate change enhances the capacity to manage other risks (Denton et. al., 2014).

4 ADAPTATION

In the context of geotechnical engineering, adaptation strategies for structures or other assets can be divided into three categories:

- (i) Reducing vulnerability. For example, designing to a higher standard helps structures withstand changes in future loading.
- (ii) Avoiding exposure. For example, this may be achieved by building in a less risky location.
- (iii) Increasing adaptability. By accommodating future change in the design this approach helps avoid locking in investments that could make future adjustments difficult and costly.

4.1 Reducing vulnerability

When assessing the cost/benefit ratio of projects in the built environment, it is normal practice to project the investment life of the project for 30 to 100 years. It is usually assumed (often implicitly) that the distribution of past variations in the environment is a reasonably accurate guide to the range of likely future variations: e.g., historical records of the variations in future river flows, coastal tidal and storm event patterns (Mazmanian et al 2013).

However, future environmental conditions will be dramatically different because of the accelerating rate of climate change and so relying on the past patterns when evaluating development proposals is hazardous.

Loadings and other design criteria specified in national legislation and standards are commonly applied when designing structures. However, these criteria often lag well behind current scientific knowledge and professional best practice as they are usually based on historical conditions and the processes to change them can be quite lengthy. These standards should be seen as setting a minimum standard and should be critically assessed by designers and clients. Clients have a vested interest in ensuring the longevity of their assets and should not assume that compliance with local or national legislation will be enough to meet their long-term requirements in a changing environment.

4.2 Reducing exposure

Reducing the exposure to future climate change risk requires an understanding of how the hazards described in Section 2.2 will affect specific sites proposed for development. This is more challenging for some hazards than others and can be more challenging on a site-specific basis than regionally.

Coastal inundation is complex, but less so than some other hazards. Forecasts of sea level rise are available, and models have been produced for many regions in the world indicating areas that are likely to be inundated under a range of climate change scenarios. These can then be considered in land-use planning and site selection. The models can be used directly in design (e.g. establishing appropriate setbacks from the inundation extent or minimum floor levels above flood levels) with an appropriate scenario selected to match local regulations and the client's risk appetite.

Other hazards are more challenging to model. For example, changes in slope stability are likely to be controlled by changes in total rainfall, changes in rainfall intensity, changes in vegetation and changes in temperature. While models are available for each of these input parameters, there are no readily available tools to assess what impact they will have on a particular slope. Collection of quality open geotechnical datasets will become increasingly important to support modelling of future impacts.

Hazards that are relatively consistent across large areas, such as settlement of expansive soils, can be readily addressed on a regional basis by identifying at risk areas and adapting building codes to be more resilient. However, codification can create unintended negative consequences such as increased construction cost and emissions.

4.3 Increasing adaptability and dealing with uncertainty in decision making

A series of decisions are made when zoning land use, developing land, making infrastructure investment, and designing. These decisions occur at many stages in the process starting with identifying the need for change through business case, concept design, site selection, detailed design, construction, operation, and decommissioning.

Models based on past patterns and events are often inadequate for characterising the effects of global warming and the complex dynamics that will result (Lempert et al, 2000). This places us in an analytical environment of deep uncertainty, which Lempert et al (2000) defines as “a condition where the parties to a decision do not know or do not agree upon the system model relating potential actions to outcomes, the prior probabilities for the value of key uncertain input parameters to the system model(s), and/or the value function that should be used to rank alternative outcomes”. A key challenge when dealing with climate change is the lack of certainty about the future timing and rate of change – we cannot easily predict the conditions that our projects will encounter during their design life.

To cope with this deep uncertainty, a host of methods are being developed. These range from simulations to narratives and Delphi and Foresight exercises (in which individuals and groups participate in imagining plausible scenarios about future states, based on alternative projections of climate change), to “no-regrets” strategies and the dynamic adaptive pathways planning approach. All these approaches are intended to help policy makers better gauge what courses of action are preferable, in the light of the deep uncertainties they face.

Dynamic Adaptive Pathway Planning is the preferred approach in New Zealand, and is described further in MfE 2017. It is a decision-making process where multiple possible outcomes are prepared for. Trigger points are defined within the design process that will, at some point in the future, determine which outcome will be followed, or at what time the next stage will take place. In geotechnical terms, this is somewhat analogous to the observational method in geotechnical engineering described by Peck (1969), but with the addition of more formally defined trigger points and subsequent actions.

For example, a building may be designed on land that is currently stable, but which may have its stability reduced below acceptable criteria under certain climate change scenarios. The designer could consider options to create more resilient foundations from the start, or instead could design a

dynamic approach where they set aside a suitable area of land for a future retaining wall to be built when groundwater hits a particular trigger level.

Adaptive management approaches, which enable actions or policies to proceed in the light of uncertainties, are not new. They have been used for resource management decision-making (e.g., water quality) and policy development both internationally and in New Zealand over the last few decades (Lawrence et al, 2020).

In response to rising sea levels around our shores, the New Zealand Coastal Policy Statement and supporting guidance (MfE, 2017) advocate the use of an adaptive planning approach to deal with the uncertainty and change around associated risks in the future. Policy 27 of the NZCPS outlines a strategy for managing the rising risk to existing coastal developments from climate-change effects, where a range of options for reducing coastal hazard risk should be assessed over “at least 100 years” and include “*identifying and planning for transition mechanisms and time frames for moving to more sustainable approaches*”.

5 MITIGATION

5.1 Reduction targets

Multiple lines of evidence indicate a strong, consistent, almost linear relationship between cumulative CO₂ emissions and projected global temperature change (IPCC 2014). To encourage mitigation, New Zealand has several greenhouse gas emissions reductions targets. Our international targets are:

- Five per cent reduction below 1990 gross emissions for the period 2013-2020
- 30 per cent reduction below 2005 (or 11 per cent below 1990) gross emissions for the period 2021-2030 (MfE, n.d.).

In addition to these international targets, in 2019 the New Zealand Climate Change Response (Zero Carbon) Amendment Act set into law a new domestic target of net zero emissions of all greenhouse gases other than biogenic methane by 2050. This new target brings New Zealand in line with the global ambition set under the Paris Agreement.

5.2 Construction sector impact

The building and construction sector is a large contributor to greenhouse gas emissions from producing materials, constructing buildings and infrastructure, and the energy used in buildings. Globally, energy use in buildings contributed 19% of society’s global carbon footprint in 2010, according to the Intergovernmental Panel on Climate Change.

Vickers et al. 2018 identified that New Zealand follows similar trends to international figures (Figure 5). Considering the full life cycle (construction, use and end-of-life), the contribution of the built environment (i.e. buildings and infrastructure) is

approximately 13% of New Zealand’s gross carbon footprint. However, this value ignores emissions generated overseas during manufacture of construction materials, many of which are imported. Adjusting for the carbon footprint embodied in New Zealand exports and imports, the contribution to CO₂ equivalent from the built environment climbs to 20%.

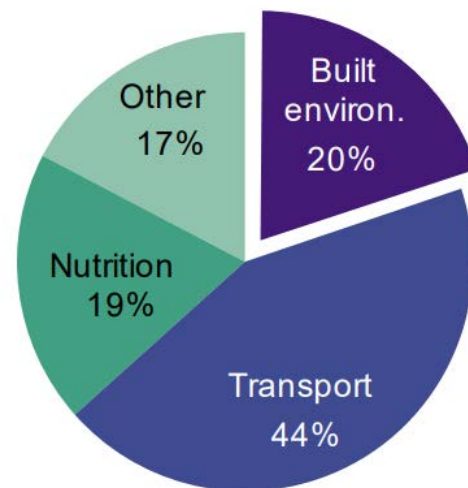


Figure 5. A breakdown of New Zealand’s carbon footprint in 2015 from a life cycle consumption perspective including international trade (Vickers et al. 2018). CO₂e is CO₂ equivalent, a measure of greenhouse gas emissions.

Emissions from buildings and infrastructure are commonly divided into embodied emissions, operational emissions, and end-of-life emissions.

Embodied emissions are the emissions generated during the manufacture of the building products and materials used in construction, maintenance and renovation. They occur upstream of the building itself, are one-off or irregular, are largely invisible to the architect or builder, and are often locked in before the first occupier even steps into the building for the first time. Given that these emissions cannot be changed later, they gain in importance over time as the energy mix used to operate the building decarbonises (reducing the relevance of the operational phase).

In New Zealand, embodied carbon for residential buildings is dominated by emissions from the production of steel and concrete (Figure 6).

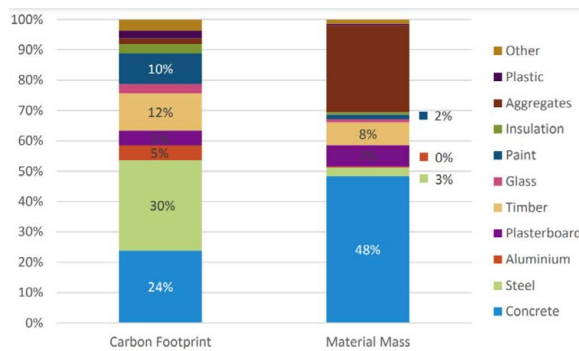


Figure 6. Carbon footprint and material mass breakdown for residential buildings in NZ over their full life (ThinkStep, 2019).

Operational emissions are the emissions produced by running a building (e.g. through heating and cooling). They are very visible as there is an ongoing cost associated with them (utility bills, maintenance bills, etc.), which creates a financial incentive for reduction. They can be improved through retrofits (e.g. replacing electric radiators with high-efficiency heat pumps) and higher-specification newbuilds (e.g. better insulation and air-tightness); however, there are cases where 'lock-in' occurs (e.g. under-slab insulation and building orientation).

End-of-life emissions are those generated in the demolition of a structure and the disposal of the resulting material.

Vickers et al. 2018 divided the emissions into these groups and identified that approximately half of all emissions were embodied in building materials (used for both buildings and infrastructure), and half were from operating our building stock (i.e., buildings only). Only a small proportion were from end-of-life. This was also supported by MBIE 2020b (Figure 7).

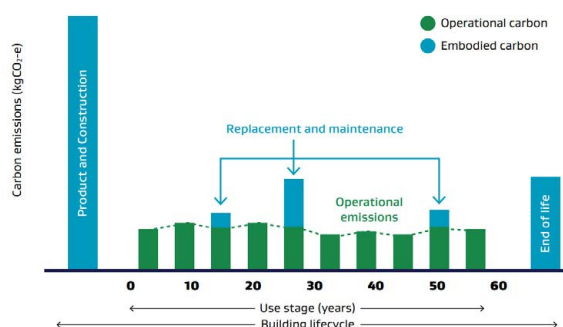


Figure 7. Operational and embodied carbon emissions over the life cycle of a building (MBIE 2020b).

However, for the unoccupied built environment (e.g. roads and other infrastructure), embodied emissions account for over 90% of the life cycle emissions (Huang et al, 2018), and therefore management of embodied emissions in infrastructure is likely to become a significant focus.

Chandrakumar et al (2019) attempted to calculate the acceptable whole-of-life carbon emissions for a residential building in New Zealand to be compatible

with a global warming limit of 2°C. They found that the climate target of a detached New Zealand house over a 90-year lifetime is 71 tonnes CO₂ equivalent, and reported that this would be equivalent to a reduction of 80% relative to current practice. Even in the best-case scenario where operational emissions were eliminated entirely, construction would still need to eliminate 60% of embodied carbon to meet national objectives. This is a very significant challenge, and one we have no current clear pathway to achieve.

5.3 Mitigation in geotechnical engineering

Significant carbon equivalent reductions are required from our construction practices in order to meet our climate change goals. However, advice on achieving these in the geotechnical realm is rare, and it has been reported that emissions from the construction industry in New Zealand have increased by 66 percent in the decade from 2007 to 2017 (NZGBC, n.d.).

Concrete and steel are among the most widely used resources in geotechnical engineering, but efficient use is not always a significant consideration in design. The global production of cement has grown very rapidly in recent years, and after fossil fuels and land-use change, it is the third-largest source of anthropogenic emissions of carbon dioxide (Andrew, 2018).

Despite this, because of the longevity of these materials, the whole-of-life carbon cost can be lower than other, initially less carbon emitting alternatives.

There are few tools to allow a robust comparison between different design alternatives in the geotechnical sphere. The BRANZ "Whole-building whole-of-life framework" provides tools, data and information to support decision making for sustainable building design. It assists calculation of the climate change impacts and other environmental impacts of our buildings (BRANZ, u.d.). This tool does allow a comparison between concrete and other building materials using data valid for New Zealand, but it focusses on structural elements, does not directly consider foundations and has no data for subgrade improvement, earthworks or other geotechnical structures.

6 NZ GOVERNMENT RESPONSE

Partly in response to our governments' commitment to UN Sustainability Goal 13, the New Zealand Ministry of Business, Innovation & Employment is developing a programme of work ("Building for Climate Change") designed to reduce emissions from buildings during their construction and operation, while also preparing buildings to withstand changes in the climate (MBIE, n.d.). Their stated goals for 2050 are:

- New Zealand's buildings are using as little energy and water as possible. They are warmer, drier and better ventilated, and provide a healthier place for us all to work and live.

- The wellbeing of New Zealanders has improved, they're leading healthier lives, and respiratory illnesses from cold and damp houses is uncommon. People also have more money in their pockets due to lower energy bills.
- Our infrastructure finds it easier to respond to demand for water, due to our lower use. This means we cope better with water shortages than we ever have before.
- The efficiencies from the Sector have made it easier for the grid to become more renewable meaning less emissions for the energy we do use.
- Energy efficiency and carbon cost are core considerations for the Sector and designs now meet an emissions budget as well as other regulatory requirements.
- Reusing buildings and recycling materials is an established part of a Sector that is well on the way to having a fully-fledged circular economy well supported by local supply chains.

In July 2020 MBIE released a consultation document describing a proposed framework for change within the building and construction sector (MBIE, 2020a). These changes, currently focused on new buildings, involve defining operational efficiency levels for buildings to be able to get building consent. Of particular relevance to geotechnical practice, they also include plans to reduce whole of life embodied carbon. The framework will set mandatory reporting requirements, and embodied carbon caps that will need to be achieved to get consent.

7 A FRAMEWORK FOR GEOTECHNICAL PRACTITIONERS

Decisions made by geotechnical practitioners, normally in consultation with their clients, colleagues, and supply chain, can have a significant impact on the ability to integrate mitigation and adaptation into a design.

Geotechnical practitioners are likely to have a requirement to change how we decide on sites for development and on design solutions in response to government policy (e.g. the MBIE Building for Climate Change programme).

The following proposals are made to assist with this decision-making process.

7.1 Assess future conditions

Without understanding how conditions affecting our project sites may change in the future, we cannot take them into account in the design. Geotechnical practitioners should take a lead in considering how climate change could affect the ground, and what this means for design and site suitability.

Consider current conditions and a range of plausible future environmental conditions that may exist during the functional life of the structure. Identify how each of these may change:

- The geological model
- Geotechnical parameters
- Vegetation cover, type and density

7.2 Question the scope

Once geotechnical professionals get involved in a project it will often have been scoped, at least at a concept phase, and the geotechnical role is limited to delivering on that scope. However, geotechnical professionals have an understanding of hazards and climate change that might not have been considered in the decisions made.

It is imperative that an understanding of the client's drivers is shared by the whole project team, and that the project scope is regularly re-visited to re-confirm that it is still the best way to solve the client's problem.

7.3 Plan for changes in climate and society

Consideration of how the long-term needs and requirements of clients will change due to the impacts of climate change is rapidly becoming a necessity. If a societal or climate change means that the functionality is partly lost, is the design flexible enough to allow easy adaptation for other likely scenarios?

Bringing climate knowledge into early planning discussions with clients is essential to identify if a more flexible approach may be advantageous for them.

7.4 Consider functional life

Traditional design focuses on providing assurance that the structure will be safe over a specified design life, with little consideration given to what happens after that point. Most structures significantly outlast their design life and so will be subject to changes that did not occur within the design life of the structure. In addition to designing for the specified design life, geotechnical engineers should consider the full functional life of a structure. If a fully functional structure has to be demolished after its design life because the designer did not consider long-term effects, this is a significant waste.

Where possible, we will need to provide better techniques to preserve existing building stock so as to avoid the negative carbon footprint of building new buildings and demolishing the old.

7.5 Use scenario-based design

Dynamic Adaptive Pathway Planning can be used to give an insight into how future changes may affect a structure. A similar approach may be used to identify the range of plausible future scenarios (using the assessment of future conditions recommended in Section 7.1).

In the case of slope stability assessment, it is already normal practice to consider a range of load cases and model each independently, often with differing

requirements for a factor of safety depending on the likelihood of the load case occurring during the design life or the uncertainty of the assumptions made.

This proposal takes the same approach and extends it to considering future climate scenarios which may alter soil properties, groundwater levels, vegetation reinforcement etc.

Scenarios that are only likely after the design life of the building could, with agreement of the client and regulator, accept a lower factor of safety than scenarios likely to occur within the design life.

7.6 Balance longevity and adaptability

We must consider both mitigation and adaptation in our designs. Often these requirements will be in conflict. For example, a very resilient foundation design may require more concrete and therefore have higher embodied emissions.

As geotechnical practitioners, we have the knowledge to offer designs that move beyond the basic 'single design' approach to a more adaptive approach that considers the longer term with a series of staged dynamic changes to ensure that the structure remains useful, resilient, and responsive to its environment.

It is recommended that a suite of options is developed, supported by clear advice to your client about the relative costs and benefits of using more robust (and potentially more carbon intensive and initially expensive) design relative to more adaptable designs which may have lower up-front costs but higher costs over their lifecycle. Consideration must be given to how to avoid the moral hazard of selecting the lower-cost option up front and leaving higher mitigation costs to future owners, particularly where the client is not likely to be a long-term owner of the building.

7.7 Optimise material use with carbon accounting tools

Although development of widely available tools for carbon accounting is still in the relatively early phases, and those that do exist are generally quite limited in the geotechnical sphere, early adoption of these techniques will drive further progress and give a more robust basis for decision making on projects. The whole-building whole-of-life framework may provide a useful starting point (BRANZ u.d.).

There are two primary options to reduce embodied carbon; increase building material efficiency, and reduce carbon intensity.

Increasing building material efficiency means using less material in new buildings, including reducing waste and minimising replacement over the building's life cycle.

Reducing the carbon intensity of the materials used in new buildings is achieved by either by making

design choices to use low-carbon materials over high-carbon alternatives, and/or reducing the embodied carbon of the construction materials (MBIE 2020b).

Both should be considered as part of the design process.

7.8 Assess sustainability throughout design

Safety in design is now becoming common practice. By considering safety from project conception through to decommissioning, safety is being much more deeply considered in design.

The same approach should be standard practice for sustainable development and climate change response. Although geotechnical practitioners are often only involved in a design process after some key decisions have been made, their role in site selection and hazard assessment gives them a rare opportunity to influence the whole design process.

Consider how the materials used in the structure could be re-used or re-purposed at the end of the structure's life, and where appropriate adjust the design to make this process easier. For example, consider how piles might be reused by future buildings, or how a slab footing could be crushed to make aggregate and what effect the reinforcement may have on this. Document these assumptions and make them available in the design documentation so that they can be used at the point of demolition.

8 ROLES AND RESPONSIBILITIES

8.1 Role of central government

In New Zealand the government has a significant unbudgeted exposure to liability for natural hazard impacts, which will increase over time with growth in the values at stake and the anticipated weather effects of climate warming and sea level rise. The mean projections suggest that the Crown's annual contingent liability for natural hazards would grow from \$0.7 billion in 2020 to \$3.3 billion in 2050. That liability could be effectively reduced by investing in natural hazard risk mitigation that reduces risks and hence liability (NZIER, 2020). While insurance and reinsurance can only cover some of the risks of losses caused by hazards, in other cases, government faces an undefined liability for reinstating infrastructure damaged by natural disaster and for providing disaster relief. For the largest events, the impacts are potentially destabilising for government finances. Governments therefore have a huge incentive, and a huge responsibility, to manage these risks.

Governments can influence the energy optimisation of private buildings, transportation and land use through their legal decisions and regulatory instruments. Central government bodies generally set minimum building standards. In New Zealand the central government role is particularly important in driving reductions in embodied carbon.

Reducing operational emissions is of direct benefit to the owner as it can result in reduced whole-of-life costs and is therefore relatively easy to incentivise. However, reducing embodied carbon in structures rarely results in lower whole-of-life costs, and is more likely to require government regulation to drive change.

8.2 Role of local government

Because local government entities are closer to their citizens than central governments, they can play a significant role in launching initiatives and bringing policies into action. However, local governments are limited by different levels of influence and control when they try to tackle global warming issues.

In New Zealand, local governments primary tool is in land-use planning to encourage adaptation measures.

Local governments can also motivate and educate their communities and stakeholders to take action to reduce carbon emissions. In comparison to national governments, local governments leverage proximity to their citizens to maximise motivation and education in this area.

Local governments have a lot of authority to reduce municipal carbon emissions in their own facilities. Although municipal buildings contribute little to overall urban emissions, the public building sector is one of the highest emissions sectors under direct municipal control. Local governments can act as role models for the private and commercial building sector by attempting to meet global climate targets.

8.3 Role of clients

Clients must start to demand buildings which are fit for the coming century and beyond. Requiring sustainability and climate change to be considered throughout the design process by consultants and contractors (in the same manner as safety in design) would make a significant difference. When assessing tenders, consider what suppliers are doing to ensure the structure will not be exposed to future hazards, and will continue to perform as intended throughout its life. Consider specifying design for an agreed emissions scenario (for example, a 2°C global temperature rise) as a minimum with an assessment required to consider the implications of a higher (e.g. 4°C) rise, and require suppliers to describe how this will change the design parameters and decisions.

8.4 Role of consultants and designers

Designers and consultants need to move beyond national standards and guidelines and start considering a range of possible climates that may exist over the real life of a structure rather than only over its design life.

Designers should discuss with their clients the risks and encourage them to embed climate change mitigation and adaptation in their brief.

9 CONCLUSIONS

Our climate is changing, and this change poses many challenges for owners and designers of structures that are usually expected to last for many decades or even centuries.

Geotechnical professionals have an understanding of how climate changes the ground on which these structures stand and can provide insight into how changes in climate may change the natural hazards to which these structures will be exposed.

Geotechnical professionals have the skills to help their clients adapt to, and to some extent to mitigate, the effects of climate change.

A framework is presented that proposes a simple approach to integrate the appropriate thinking process into design.

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Comparing the embodied carbon and local environmental impact of common geotechnical foundation solutions for the Australian market

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ABSTRACT

In the Australian geotechnical market, foundation solutions for construction projects are commonly assessed against three key criteria; cost, program and quality. Safety and environmental aspects are often not assessed in the process until a geotechnical foundation technique has been selected based on the aforementioned criteria. As climate change has progressed to become one of the largest and most pertinent environmental issues in society today, it has in turn brought sustainability to the forefront of post construction assessments and is now a greater focus for government bodies, private developers and wider stakeholders. Using Keller's inhouse carbon calculator and a series of environmental metrics, this paper aims to inform decision makers of the carbon emission and social impacts of various geotechnical foundation solutions prior to their selection. Two theoretical projects are used to compare a range of geotechnical solutions: a 60-storey high-rise building requiring heavy foundations, and a low load warehouse building where ground improvement solutions can be used. Each geotechnical solution designed for the two Australian projects is then assessed using a standard cradle-to-gate carbon calculator. In addition, the impact on the local community is assessed according to the noise, traffic and emissions generated by each solution.

Keywords: Foundations, Piling, Ground Improvement, Carbon, CO₂e, Emissions

1 INTRODUCTION

The construction sector has many impacts on local environmental sustainability, from piling machinery generating noise and dust to the traffic congestion caused by bringing materials on to site. The construction sector also produces 39% of Global Greenhouse Gas (GHG) emissions (IEA, 2020 & UNEP 2019); foundations and other substructures can be responsible for between 13% to 30% of these construction emissions (WRAP 2019, Menzies, 2011). This makes geotechnical contractors and their supply chains key to meeting global emissions goals, such as the Paris Accord and Climate Action (Goal 13) of the UN Sustainable Development Goals.

Nationally, the creation of the Green Building Council of Australia (GBCA) in 2002 aims to incentivise delivering projects with severely reduced, or even positive, environmental impacts. The GBCA is driving this change with a vision "to lead the sustainable transformation of Australia's built environment" and targeting a net zero emissions outcome for new buildings by 2030 and for existing buildings by 2050.

GBCA assigns green star ratings to buildings based on their design, construction and operation meeting sustainability criteria. The GBCA assessment also assigns credits for a whole of site life cycle assessment. Much like BREEAM (Building Research Establishment Environmental Assessment Method) in Europe or LEED (Leadership in Energy and Environmental Design) in America, these green star ratings are increasingly valued by developers, customers, investors and clients alike.

One of the best ways geotechnical contractors can reduce their sustainability impacts and achieve these green building accreditations is by changing the geotechnical solutions they offer clients. Yet currently, geotechnical foundation solutions are

often assessed by three major categories: cost, time and quality. Safety is usually assessed and managed in detail after selection of a solution. Typically, little to no consideration is given to environmental impact of proposed geotechnical foundation solutions during appraisal of proposals.

This paper explores the prediction of carbon emissions and local environmental impact of these solutions; through the use of realistic, Australian-focused examples, aiming to highlight the impacts of different geotechnical solutions. Ultimately, geotechnical companies can then integrate carbon calculations and social sustainability into client's project assessment and decision-making as standard. In turn, this can act as a driver for the adoption of low carbon, low impact geotechnical solutions, where there is client demand. Foundation contractors can also then be measured against these predictions, as they are against other categories.

2 METHODS

2.1 Emissions Calculator Overview

The calculation of geotechnical solution carbon emissions uses Keller's inhouse carbon calculator. This uses the same system boundaries and carbon intensity multipliers as published industry standards, such as the European Federation of Foundation Contractors - Deep Foundation Institute (EFFC-DFI) calculator, which is in-keeping with the GHG protocol of the World Resources Institute (WRI) and the World Business Council for Sustainable Development (WBCSD). However, this Keller calculator uses direct machinery efficiency and supply chain data from Keller Australia and their supply chain. This 'cradle-to-gate' assessment incorporates everything from the extraction and manufacture of the raw materials through to the final installation of the foundations (Figure 1) and ensures all three carbon emission scopes are considered (WRI & WBCSD, 2004), as set out below:



Figure 1. 'Cradle-to-gate' life cycle of the carbon calculator.

- Scope 1 – Direct (or point source) emissions, such as fuels burnt and oils used on site.
- Scope 2 – Indirect emissions from electricity obtained from the utilities power grid for the site concerned.
- Scope 3 – Other indirect emissions embodied in the materials use and commuting of materials, equipment and personnel to site.

Activity data (such as consumed fuels and materials) are the primary input for determining GHG emissions. These activity data are multiplied by known factors for a given product, vehicle, equipment or machinery to determine the GHG emissions. GHG emissions are dealt with collectively using CO₂ equivalent (CO₂e) capturing effects of CO₂, CH₄ and N₂O, according to the GHG protocol global warming potential multipliers for CO₂e.

For most geotechnical projects overall emissions are primarily dictated by Scope 1 and Scope 3 whilst Scope 2 emissions represent a minimal contribution to the overall emissions.

Primary emission sources assessed are material manufacturing and transport, consumed energy, equipment transport, equipment manufacturing waste treatment and transport, personnel transport.

Secondary emission sources (such as depreciation/life span of machinery, steel transport) is generally assessed using a standard database of ratios for secondary emissions, unless activity data can be accurately and confidently estimated for the project. For the purposes of the comparisons within Section 4, life span of machinery has not been assessed.

Most factors have been adopted from Australian Government National Green House Accounts (NGA) Factors (2018). A full list of databases referenced for emissions factors can be presented upon request to the author.

2.2 Local Environmental Impact

Local environmental impact is assessed subjectively and comparatively via a combination of metrics. These metrics are intended to allow assessment of project foundation solution impact upon the community local to the project site. These metrics include:

- Emissions generated on site.
- Spoil or material waste sent to landfill.
- Noise generated on site, measured via equivalent continuous sound pressure (LA_{eq}) and 90th percentile noise level (L90).
- Vehicle movements/arrivals on site, total and daily, providing an indication of effect on local traffic conditions. Assumed distances for data calculations in Section 4 were between 20 to 50km based on recent project data.

LA_{eq} and L90 give an indication of noise levels on site for a standard workday. For consistency all noise levels are measured at 30m from source. 30m has been selected on the assumption this distance for most sites in Australia is a good representation of average distance from noise source to site boundary or sensitive neighbours.

The amount, reuse of and contamination of any waste generated will be influenced by site variables such as existing contamination, soil density/porosity and previous site use. For the purposes of the comparison in Section 4, it has been assumed that 50% of the cement-based and steel waste is recycled with carbon emissions from the recycling attributed to the project those recycled materials are to be used on, i.e. not captured within this modelling presented in Section 4. The remaining 50% is assumed to be inert waste sent to landfill.

Drilled soil and rock can also be reused for certain ground conditions. However, for the scenarios assessed in Section 4 it is assumed the ground

conditions are not suitable for spoil reuse and all spoil is sent to land fill.

3 THEORETICAL PROJECT DETAILS

3.1 High-Rise Building Foundations

Assumed high-rise building details are 60 storey, footprint of 2,500m², building load 1,650MN or an equivalent pressure of 660kPa.

The high-rise building foundations were all designed to support the same floor and core loadings to represent equivalent schemes. Details of each foundation scheme considered are provided in Table 1. The details of each scheme, including concrete grades, reinforcement tonnages, site deliveries and machinery on site was input into the calculator.

Table 1. High-rise building foundation details

Foundation Scheme	No. of piles	Volume of works (m ³)	Days on Site
A - CFA Piles	147	2,909	58
B - Bored Piles	91	2,583	75
C - Precast Piles	691	2,762	50

3.2 Warehouse Building Foundations

Assumed industrial warehouse details are footprint of 300m x 100m = 30,000m², floor load of 80kPa with four rows of concentrated column loads:

Details of each foundation scheme considered are provided in Table 2. It should be noted that many of the solutions presented in Table 2 are not mutually compatible in all soil ground conditions and each provide a different settlement performance for the structure from 5mm through to 150mm. For the purposes of this comparison it has been assumed that each technique is applicable for the hypothetical ground conditions. It should be noted that scheme H would rarely be considered as a viable solution unless there are significant onsite restrictions.

Table 2. Warehouse building foundation details

Foundation Scheme	Number of elements	Volume of works (m ³)	Days on Site
D – CFA Piles	1,450	8,200	107
E – Bored Piles	1,450	8,200	100
F – Precast Piles	1,450	4,571	107
G - Rigid Inclusions	5,800	7,617	45
H - Jet Grouting	3,734	54,900	134
I – DSM [#]	3,734	54,900	85
J – VSC*	5,800	37,901	111

*Vibro Stone Columns, [#]Deep Soil Mixing

Schemes were assessed based on most realistic resourcing typically provided in the Australian market whilst targeting programs regularly required by clients.

4 RESULTS

4.1 High-Rise Building Foundations

Calculation results presented in Table 3 demonstrate Scheme B generated the lowest amount of CO₂e emissions with schemes A and C each generating 85t and 367t more emissions respectively.

Table 3. CO₂e Emissions for high-rise building foundation schemes

Metric	CO ₂ e Emissions (kg/MN)	Total CO ₂ e Emissions (tonnes)
A – CFA Piles	1,017	1,679
B – Bored Piles	966	1,594
C – Precast Piles	1,189	1,961

The source of emissions is presented in Figure 2, demonstrating that the overwhelming majority of CO₂e emissions is due to materials manufacture. The next highest CO₂e emissions are a result of onsite equipment whilst the remaining categories make up less than 5% of the CO₂e emissions.

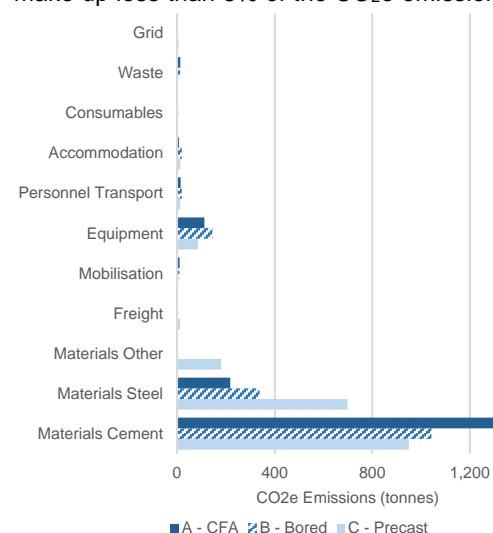


Figure 2. CO₂e Emissions by category for high-rise building foundation schemes

Table 4 presents the local environmental impact metrics for the three schemes investigated. The author acknowledges that the priority placed on each environmental metric will vary with the site setting. Therefore, it is not practical to draw a definitive preference out of the results presented given most sites are unique when consideration is given to neighbouring structures, community, traffic network and background noise. Nonetheless, this comparison still allows us to highlight key differences between solutions, which can then be considered in each local environment.

As expected, noise generated is higher for scheme C, 88dB compared with 84dB for the other two schemes. Time on site is significantly less for scheme C, demonstrating an 8 and 25 day advantage over schemes A and B respectively.

Table 4. Local Environmental Impact Metrics for high-rise building foundation schemes

Metric	A CFA Piles	B Bored Piles	C Precast Piles
CO ₂ e Emissions on Site (tonnes)	112.9	145.6	86.2
Total site vehicle arrivals	1,233	1,226	817
Total site vehicle arrivals per day	21	16	16
LA _{eq} @ 30m from machinery (dB)	84dB	84dB	88dB
L90 @ 30m from machinery (dB)	77dB	77dB	79dB
Days on site	58	75	50
Waste, spoil (tonnes)	5,236	4,649	0
Waste, materials inert (tonnes)	437	388	117

Schemes A and B require the dumping or reuse of a significant amount of extracted spoil waste. However, scheme C, as a driven precast pile scheme, displaces the ground and therefore does not typically require any spoil removal. The environmental impact of this waste is significantly affected by the contamination of the spoil waste, but this is dictated by initial ground conditions rather than the geotechnical solution.

Inert waste for the schemes is significantly more for the in-situ concrete Schemes A and B. This is due to uncertainty around constructed pile volumes due to over drilling and over ordering of concrete. With Scheme C, precast material quantities are known, hence there is significantly less waste.

4.2 Warehouse building foundations

Table 5 shows schemes H and I had the greatest overall CO₂e emissions, eclipsing other schemes by more than 20,000t and 5,000t respectively. This is due to high volume of treatment zones and high cement consumption for each scheme. Interestingly scheme J resulted in significant CO₂e emissions due to a large use of aggregate despite lack of cement or steel content.

Of the remaining schemes, G had the lowest estimated CO₂e emissions with 2,610t. This is attributed to low relative treatment volumes, low cement consumption and efficient onsite plant, though it should be noted a greater tolerance of overall structure settlement is required.

For the 3 piled schemes assessed, scheme F had approximately 1,000t less CO₂e emissions than schemes D and E, primarily due to lower volume of materials required. This is contrary to the high-rise schemes A, B and C. The reason for this is in this scenario geotechnical capacity governed the design details and accordingly for schemes D and E pile structural capacity is not fully utilised whilst for scheme F, the piles utilised most of their structural capacity.

Table 5. CO₂e Emissions for warehouse foundation schemes

Foundation Scheme	CO ₂ e Emissions (kg/MN)	Total CO ₂ e Emissions (tonnes)
D – CFA Piles	1,745	4,189
E – Bored Piles	1,923	4,615
F – Precast Piles	1,367	3,281
G - Rigid Inclusions	1,088	2,610
H - Jet Grouting	11,398	27,356
I – DSM	4,224	10,138
J – VSC	1,870	4,489

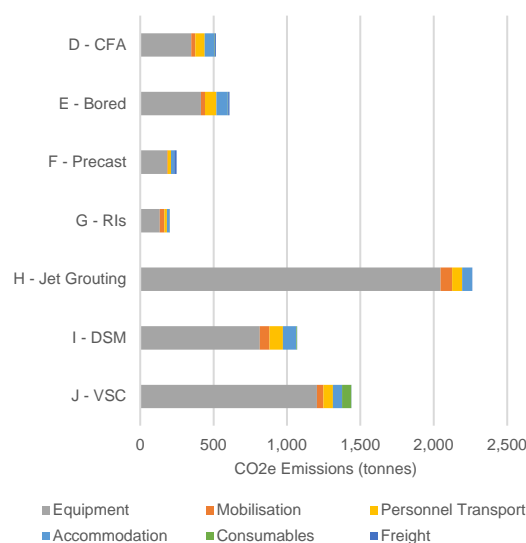


Figure 4. Scope 1 CO₂e Emissions for warehouse foundation schemes.

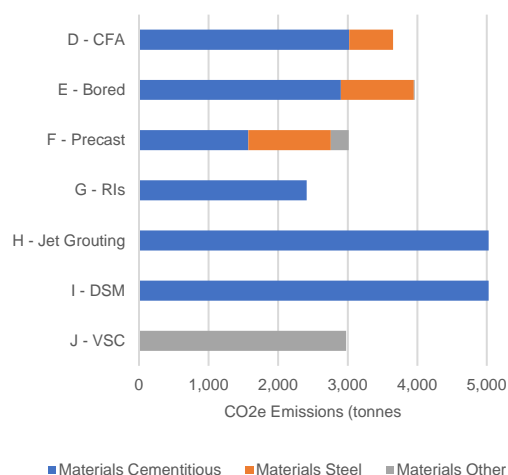


Figure 5. Scope 3 CO₂e Emissions for warehouse foundation schemes.

Graph truncated for clarity, extent of I – DSM (9,053t) and H – Jet grouting (25,039t) not shown.

The warehouse schemes cannot be solely assessed by CO₂e emissions for geotechnical works. The schemes require varying solutions for superstructure, primarily ground slab, ground

Table 6: Local Environmental Impact Metrics for warehouse foundation schemes.

Metric	D CFA Piles	E Bored Piles	F Precast Piles	G Rigid Inclusions	H Jet Grouting	I DSM	J VSC
CO ₂ e Emitted on Site (t)	351	418	184	135	2050	818	1210
Total vehicle arrivals	4,278	4,786	952	2,395	10,824	5,981	6,849
Vehicle arrivals per day	40	48	9	53	81	70	62
LA _{eq} @ 30m (dB)	85	86	88	89	89	92	88
L90 @ 30m (dB)	79	79	79	79	79	82	79
Days on site	107	100	107	45	134	85	111
Waste, soil/rock (t)	14,760	14,760	-	686	49,410	19,764	3,411
Waste, cement/steel (t)	1,232	1,183	552	914	6,588	6,588	4,548

beams and pile caps. Scheme G, for instance, will result in an on-ground slab with placement of a load transfer platform whilst schemes D through F will require a suspended slab with ground beams. To robustly calculate CO₂e emissions, for each scheme, superstructure through to top of ground slab level should be assessed in combination with the presented Figures 4 and 5. However, this is beyond the scope of the cradle-to-gate carbon calculator methodology and therefore rests with the main contractors, project managers or consultants to assess.

Scheme G was the lowest onsite CO₂e emissions of 135t due to efficient onsite equipment and low number of days on site. Scheme F is next with 184t of CO₂e emitted onsite primarily due to fabrication of piles off site. The highest onsite CO₂e emissions were from schemes H and J due to a large amount of equipment onsite for a lengthy period compared to the other schemes.

The L90 noise levels for each scheme is relatively consistent between 79 to 82 dB each. LA_{eq} levels were lowest for schemes D and E with 85dB and 86dB respectively. Scheme I had the highest LA_{eq} at 92dB, surprisingly, larger than the driven precast pile scheme F at 88dB. This was primarily due to many compressors, rigs and other ancillary equipment onsite compared to scheme F, precast piles, which utilised louder but a significantly lower number of equipment.

The waste generated for scheme H was by far the greatest due to large treatment volumes whilst schemes D, E and I also generated significant waste requiring to be managed.

5 DISCUSSION

It can be inferred from the calculations that best reduction of CO₂e emissions for foundation schemes can be achieved by targeting reduction in material tonnage and time on site. Auspiciously, reduction of these categories directly reduces project costs, accordingly projects are already incentivised to reduce them as far as is practical.

Australian design codes for reinforced concrete structures and piling design utilise a limit state design approach. Adopting a higher factor of safety

for design directly results in use of additional materials, either by increased cement content, steel content or longer period spent on site. Unnecessary conservatism in design can therefore lead to increased CO₂e emissions, particularly where structural capacity of the solution is not being fully utilised.

It should be noted the recent release of the Australian Concrete Structures code AS3600-2018 has increased reduction factors by 5% to 8%, accordingly a similar, if not slightly lower, reduction on CO₂e emissions will be realised as implementation of this code becomes widespread.

In general, the longer the time spent on site, the higher the Scope 2 CO₂e emissions of the project. A significant reduction in vehicle arrivals/deliveries is achieved via prefabrication of foundations offsite. It should be noted this may not equate to a reduction in overall emissions subject to details of off-site manufacture. Off-site manufacturing also leads to significantly less disruption to local traffic throughout project durations and reduction in local air pollution reducing nuisance to people local to the site.

For many projects, stakeholders such as councils, clients and neighbours view construction noise as a key concern which can drive implementation of restrictions around construction hours and techniques leading to increased costs. Accordingly, consideration must also be given to the duration of the works as it relates to the period of disruption to neighbouring persons which must be assessed with the intensity (LA_{eq}/L90) of the disruption.

For the schemes compared within this study, the average CO₂e contribution for materials is 87.4%. Calculating CO₂e emissions for materials involves the lowest margin for error as a result of significant work done by the Australian government for National Greenhouse factors and work done by Australian materials suppliers.

Assessing the total amount of materials to be used on site is generally quite robust. As improper estimation of material use has a direct and significant impact upon cost overruns, for the majority of projects materials are estimated within 5 to 10% of actual final quantities.

The largest margins of error in the predictions likely lie within the onsite CO₂e emissions assessment. It is unlikely to be possible to accurately predict specific plant, fuel consumption and time on site for all equipment at design phase. Post construction however this information can be easily vetted by inspection of fuel consumption on site and compared to initial estimates.

The remaining categories comprise minor percentages of overall CO₂e emissions and accordingly assessing margins of error for them is not significant for assessment of the schemes.

It is estimated the accuracy of the estimations is approximately +/-12%. This is driven largely by estimated 10% and 30% margin of error for predicted vs actual material and onsite CO₂e emissions respectively.

6 CONCLUSIONS, SUMMARY AND FURTHER WORK

The carbon calculator aspect of these case studies demonstrates that material procurement and consumption have by far the greatest impact on CO₂e emissions for a given foundation scheme. For example, 5m³ to 8m³ of concrete waste represents 1,700kg CO₂e, the equivalent of a car travelling 7,000 km (an approx. distance travelled by an employee commuting 30km/day in a work year). This emphasises the size of scope 3 emissions compared to scope 1. Therefore, unsurprisingly in both case studies, those geotechnical solutions that reduce cement and steel content, or rely solely on ground improvement, have a lower carbon footprint and will assist in higher green star ratings from the GBCA.

Development of standards and specifications and factors of safety therein has a direct impact on material use. Unnecessary conservatism in design will substantially increase carbon emissions. Load testing of foundations often leads to more efficient designs and this should be favoured both for verification and emissions purposes.

In some cases, the avoidance of concrete to reduce embodied carbon is not feasible. In the high-rise case study, for example, all the potential solutions require concrete. This highlights the need for further development, research and application of low carbon cement technologies. Within Australia, this technology faces a number of barriers, such as a lack of historical data, a lack of supply and availability of low carbon cement materials and a lack of relevant standards, all of which could result in hesitance to adopt low carbon technologies.

Similarly, it is worth reiterating that not every ground improvement or heavy piling solution is suitable for all ground conditions. When coupled with local environmental factors, especially for urban areas where avoiding congestion and noise pollution is particularly important, this emphasises the need for a framework to determine what is the most sustainable geotechnical solution for each

project. In understanding and comparing the carbon and local environmental impact of these solutions, this paper aims to pave the way for this development.

In terms of further work, this paper does not touch upon the carbon or environmental impact of foundation reuse from previous buildings. In Australia, the end of life phase for foundations is typically waste, being left in-situ, often partially demolished over the top few meters to enable subsequent redevelopment of the site with new foundations installed. Although currently inhibited by compliance with new codes and standards, such as "AS2159 – Piling design and Installation", the reuse of foundations is worth exploring, both as a way to reduce carbon and reduce the impact on the local environment. Expanding AS2159 to provide guidance on foundation reuse would be a key step towards this.

This paper and the calculator do not assess the positive impact of energy piles. The next step for full assessment of foundation solutions is to incorporate this positive contribution to provide a more robust assessment of a foundation scheme carbon emissions.

The limits of this paper are acknowledged in terms of quantifying the environmental impacts of different geotechnical solutions. Nonetheless, it is hoped this comparison illustrates the relative embodied carbon and local environmental impact for various techniques commonly adopted for high-rise and warehouse foundations within Australia.

7 ACKNOWLEDGEMENTS

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Energy driven piles in Australia: Design and construction lessons from a trial at Fishermans Bend

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ABSTRACT

About 50% of the energy bill of buildings arises from space heating and cooling (air conditioning). The associated greenhouse gas (GHG) emissions of the sector account for between 14% and 25% (or higher, particularly overseas) of the total emissions. Geotechnical engineering designers and contractors have an opportunity to contribute to a more sustainable future. Shallow geothermal technology for efficient heating and cooling represents one such opportunity. Through a partnership between Wagstaff Piling and The University of Melbourne, the design and construction of the first energy driven piles in Australia was undertaken in 2019, with the last field thermal performance testing completed in January 2020. These energy piles can be connected to a geothermal system. This paper discusses the construction and installation of this small-scale field trial at Fishermans Bend (Victoria), comprising three different energy driven pile configurations. An investigation into the thermo-mechanical efficiency of driven energy piles to evaluate their capacity to provide heating and cooling for buildings is undertaken. Peer-reviewed literature already exists discussing shallow geothermal energy systems and bored energy piles. However, there is a significant gap in the literature considering driven energy piles specifically, and no public guidance about construction; thus, we aim to start redressing these issues herein. Experimental data collected by running Thermal Response Tests (TRTs) on selected energy driven piles at Fishermans Bend are presented. The information collected from the fieldwork will be used to validate detailed Finite Element Method (FEM) models and to optimise construction, minimising costs and to demonstrate none or minimal program delays.

Keywords: Geothermal energy, driven piles, innovation in construction, energy foundations

1 INTRODUCTION

The need to create and continually develop sustainable energy solutions has never been more important. Shallow geothermal energy technology provides an opportunity for small to large scale developments to lower greenhouse gas (GHG) emissions by supplementing their demand from the electricity grid. Shallow geothermal or ground source heat pump (GSHP) systems use the ground as a heat source or sink to efficiently heat and cool buildings, given that the ground temperature a few tens of metres below the surface is relatively constant throughout the year (Johnston et al. 2011).

The systems typically consist of a GSHP connected to ground heat exchangers (GHEs) and a heating and cooling distribution system in the building that they condition. Under most Australian climatic conditions, plain water is circulated through the High-Density Polyethylene (HDPE) or cross-linked polyethylene (PEX) piping to exchange heat with the ground (i.e., ground heat exchanger or GHEs). In winter, relatively cold water circulated in the geothermal pipes extracts heat from the ground. The GSHP extracts and upgrades this heat to deliver space heating (and in some cases hot water) to the building. The water cooled down by the GSHP is then reinjected to the GHEs to repeat the cycle. In summer, the system is reversed, with heat rejected to ground to provide air conditioning to buildings. For every kWh used to drive the GSHP compressor, between 4 and 6 kWh of thermal energy is delivered to the building throughout the year. This highly efficient system renders flattened and lower energy use and GHG emissions associated to the sources of

energy that otherwise would be used (Aditya & Narsilio, 2020).

One way of harnessing geothermal energy is by the use of *Energy Piles* rather than purposely used borehole ground heat exchangers in vertical systems (or trenches in horizontal systems) to significantly lower capital costs of GSHP systems (Lu & Narsilio, 2019; Makasis, Narsilio, Bidarmaghz, Johnston, & Zhong, 2020). These piles can be constructed in a variety of ways in either precast piles or bored piles. Prior to casting concrete, a series of HDPE closed circuit pipes are installed and fixed to the cage. The pipes provide a path for a heat transfer fluid to circulate and capture heat energy from the surrounding ground.

This paper specifically looks at the construction and installation of conventional jointed precast piles with embedded HDPE pipe loops. Three different configurations of precast energy piles were trialled to identify heating/cooling capacity and cost/programming implications. The field trials were conducted at a site located in Fishermans Bend, Victoria. Thermal Response Tests (TRTs) results to indicate the effective thermal conductivity of the site and thermal performance of various pipe and pile configurations are also presented.

2 SITE DESCRIPTION

The trial site is located within Fishermans Bend, the largest urban redevelopment district in the country. Ground conditions at the field trial site comprise 1.5 m of Fill, 4 m of Port Melbourne Sand, 3 m of Coode Island Silt, 13 to 14 m of Fishermans Bend Silt, 15 m of Moray Street Gravels underlain by

Werribee Formation clays (Figure 1). The groundwater table is relatively shallow and encountered around 2 to 3 metres below ground surface. Precast piles were driven at this site, details of the pile configurations are included in Section 3.

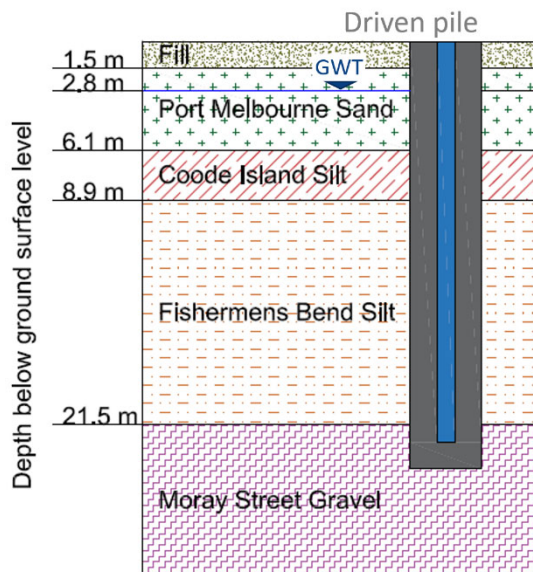


Figure 1. Soil profile (pile width not to scale)

3 PRECAST PILE CONFIGURATIONS

Precast reinforced concrete pile segments of 12 m in length and square cross section of 350 mm x 350 mm were used in this trial. Two segments were driven to form a floating pile primarily to test constructability, speed of installation, and thermal performance. Note that for structural stability, three segments rather than two would have been typically utilised in this site to reach and embed part of the pile in the firmer deeper formation. Similar results, for structural stability, may be achieved by utilising two longer precast segments (max 14-15 m). The trial precast piles were installed at this site using a Banut 700 driving rig equipped with a 6t hydraulic drop hammer.

The precast energy pile trial consisted of two pile types, namely Pile A and Pile B, and tested three geothermal pipe configurations, as schematically depicted in Figure 2. In all cases, High-Density Polyethylene (HDPE) pipe with an outer diameter of 25 mm and SDR of at least 11 has been used to form geothermal U-loops. Each U-loop was manufactured by using electro-fused fittings joining straight pipes at the bottom of the pile.

Pile A configurations include two geothermal U-loops placed inside the hollow pile segments, a cylindrical 100 mm diameter void at the centre of the pile typically used to reduce weight for transport and concrete (ultimately costs) while maintaining strength and structural functionality. Given the high groundwater table, the void is naturally filled with (ground) water ensuring no air gaps between the HDPE pipes and the surrounding concrete and soil, this is important for heat transfer.

Pile B configurations are “solid” piles with reinforcing steel cage and bars. In Pile B type, two HDPE pipe

configurations were tested. The bottom precast pile segment in the pile, the one driven into the soil first, contains U-loops cast within the pile. In one pipe configuration, the U-loop pipe length extends to the surface from the side, near the top of this bottommost installed segment. This pipe excess length is rolled in the workshop to then being unrolled on site as the second segment is driving into the ground (solid pipe lines in the schematic in Figure 2). In the other pipe configuration tested, straight pipes are embedded in the second segment with chick pipe connectors on the splicing ends of the pipe segments, such that when the dynamic splicing of the pile segments takes place, the splicing of the pipes does so as well. This pipe configuration in Pile B type requires careful placement of pipes and connectors in the segment pile heads and segment pile splicing joints while precasting and installing. Once driven, the geothermal pipes run within the bottom pile segment and then run along the outside of the top pile segment (dashed pipelines in the schematic of Figure 2).

The various pile and pipe configurations allow testing different geothermal pipe circuitry (e.g., a single pile with a single U-loop of ~40m total pipe length, or pipes from adjacent piles to be connected in series, significantly increasing total pipe length between each inlet and outlet). The centre-to-centre spacing between the two piles is approximately 2 m.

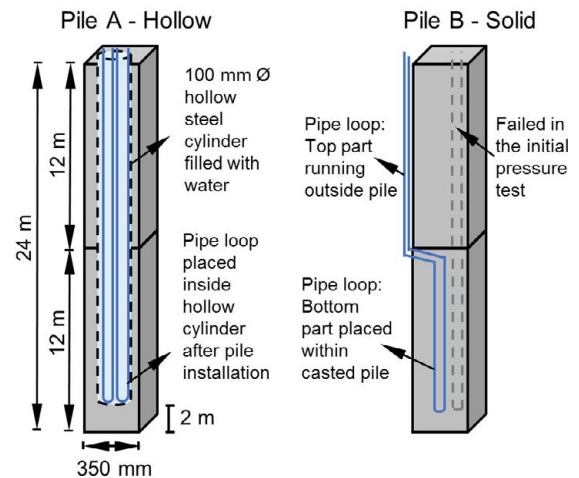


Figure 2. Schematics of the precast piles

4 THERMAL RESPONSE TESTS

4.1 In situ thermal test analysis methods

A critical parameter in designing ground heat exchangers (GHEs) of GSHP systems is the ground thermal conductivity, λ_g . The confidence in this parameter substantially influences the sizing of the GHEs and the thermal performance of the GSHP system (Mikhaylova, Johnston, & Narsilio, 2016). In-situ Thermal Response Test (TRT) is considered as the most reliable approach to obtain the site-specific ground thermal conductivity. TRTs were initially developed for traditional borehole GHEs and have been subsequently extended to the analysis in energy piles (Jensen-Page, Loveridge, & Narsilio,

2019; Spittle & Gehlin, 2015). A TRT involves heating a fluid at a constant rate of heat injection and circulating the fluid in the ground loops in a prototype GHE for around 48 hours. The inlet and outlet temperatures are constantly monitored over time, as well as the flow rate and power injection. There are several commonly adopted analytical models available to evaluate the effective λ_g from the measured TRT fluid temperatures including the Infinite Line Source Model (ILSM) (Ingersoll & Zobel, 1913; Jaeger & Carslaw, 1959), the Infinite Cylindrical Source Model (ICSM) (Yener & Kakac, 2018) and the Finite Line Source Model (FLSM) (Claesson & Javed, 2011; Lamarche & Beauchamp, 2007). Among these models, the ILSM is the most popular approach which assumes an infinite homogeneous ground subjected to a constant radial heat flow from a linear heat source. This is a close approximation to borehole GHEs, where the ratio of length (depth) to diameter is large but may require adjustment for less slender energy piles (see for example, Jensen-Page et al., 2019).

The average fluid temperature T (the average of inlet and outlet temperatures) over time can be simplified to a linear-log relationship as:

$$T(t) = \frac{Q}{H} R_b + \frac{Q}{4\pi\lambda_g H} \cdot \left[\ln\left(\frac{4\alpha_g t}{r_b^2}\right) - \gamma \right] + T_0 \quad (1)$$

where t is time [sec], Q is the applied heat power [W], H is the buried depth of GHE in [m], R_b is the GHE thermal resistance [m·K/W], α_g is the thermal diffusivity (a function of thermal conductivity, heat capacity and density) [m²/sec], r_b is the radius of GHE [m], T_0 is a farfield reference temperature and γ is Euler's constant (0.5772). Then the effective λ_g can be estimated from the gradient m of the semi-log plot of the linear relationship between the average fluid temperature T and time t as:

$$\lambda_g = Q / (4\pi m H) \quad (2)$$

Prior to commencing the TRTs, a few preliminary tasks must be performed. The individual ground loops must be firstly flushed and purged ensuring a minimum fluid velocity of 0.6 m/s (2 ft/s) through all piping for at least 20 minutes and until no air bubbles are detected, to remove any debris and air pockets. Pressure tests are conducted to ensure there was no leakage in the loops prior and after installation as well.

4.2 TRT tests in this trial

For this particular trial, a manifold with manual valves and quick clamp fittings were introduced to allow a variety of different loop combinations to be tested. A total of eight tests were performed, four of which are reported in this work. The in-situ tests were undertaken using a Precision Geothermal GeoCube unit, as shown in Figure 3. The pipes and fittings were well insulated and covered to minimise (thermal) disturbance from the environment.



Figure 3. In-situ TRTs set up

The four different piping configurations considered in this study are shown in Figure 4. TRT1 was conducted on a single U-loop embedded within one Pile A, TRT2 was conducted on two U-loops connected in series within Pile A, TRT3 was conducted on one U-loop within Pile B. In TRT4, one U-loop in Pile A with another U-loop in Pile B were connected in series, resulting a common joint inlet and outlet to be connected to the TRT unit. During the TRTs, a completed testing cycle consisted of 3-4 days of a heating phase (rejecting heat to the ground, emulating air conditioning a building) and an equivalent amount of time in a recovery phase. During the heating phase, a 1.5 kW heating element was used for TRT1 through TRT3, and a larger heating power of 3.5 kW was used for the TRT4 considering the circuit went through two different piles. During the recovery phase, the heating elements were switched off and only the circulation pump was left running to help the ground to restore to its undisturbed equilibrium temperature.

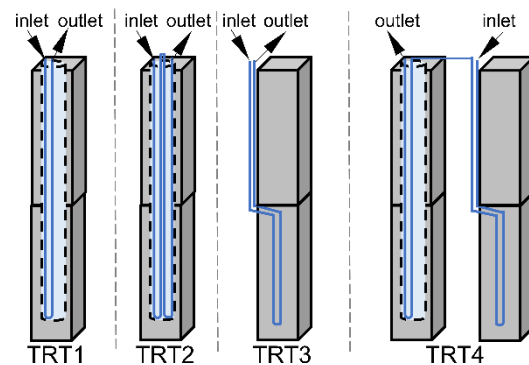


Figure 4. Pipe configurations tested with TRTs

5 NUMERICAL SIMULATION

The long-term thermal potential of the precast energy piles can be assessed through simulations. While specialist shallow geothermal commercial software like GLD or GLHEPro or more generic TRNSYS models may be used for concept design, they are unable to capture in detail the energy pile thermal behaviour. To redress this shortcoming, detailed 3D finite element models built in COMSOL Multiphysics and developed by The University of Melbourne's Porous Media Research Laboratory are used herein to explore some few cases (Bidarmaghz, 2014;

Makasis, 2018). The model couples the governing equations of the heat transfer and fluid flow. The thermal analyses consider a combination of the conductive heat transfer in the pipe wall, pile concrete and surrounding ground and the convective heat transfer in the flowing fluid (water) within the pipes. A geo-mechanical model is not included in this work. For piles driven to the firmer underlying layer, thermo-mechanical effects are known to be negligible (this may not be the case for floating piles in normally consolidated clays though). Relevant material properties used in the models are summarised in Table 1.

Table 1: Material properties adopted in the numerical simulation

Description	Value
Thermal conductivity of ground (W/(m·K)) λ_g	2.4 ^a
Density of ground (kg/m ³) ρ_g	1,940
Specific heat capacity ground (J/(kg·K)) $C_{p,g}$	1,500
Annual average ground temp (°C) T_{field}	18
Thermal conduct. of concrete (W/(m·K)) λ_c	2.0
Density of concrete (kg/m ³) ρ_c	2,400
Specific heat cap. of concrete (J/(kg·K)) $C_{p,c}$	1,000
Thermal conductivity of water (W/(m·K)) λ_w	0.58
Density of water (kg/m ³) ρ_w	998
Specific heat cap. of water (J/(kg·K)) $C_{p,w}$	4,185
Pipe flow rate (L/min) m_w	6

^a from TRT results shown later in Table 2

One pile group is modelled, consisting of 24 and 36-metre-deep precast piles, 2 metres apart, i.e., floating and embedded piles. Symmetry boundary conditions are prescribed on the far boundaries of the model to allow the modelling of only one pile group and save computational time. The undisturbed ground temperature of 18°C is applied on the bottom surface of the model while thermal insulation is applied on the top surface of the model to render zero thermal flux through the surface considering the ground is covered by the building.

The thermal heating and cooling load distribution is a critical factor for the GSHP system performance. For simplicity, a constant cooling load (-ve) is applied to the energy pile group for half year (October to March) and the same amount of heating load (+ve) is prescribed for the rest of the year (April to September). The thermal potential of the system is determined as that corresponding to the resulting maximum or minimum average fluid temperature reaching the design operational limits of the GSHP system. Typical lower and upper design limits, 0 °C and 35 °C respectively, are adopted in this work while noting that these limits may change depending on the selected GSHP model type and design.

6 RESULTS AND DISCUSSION

Both pile types were installed without issues. In a full scale roll out, the pilot test showed that Pile A type execution falls outside the critical path but may require a separate crew for piping installation after

protruding pile heads are cut on the surface, while Pile B type may add between 5 to 15 minutes per complete pile installed (3 segments to firm layer), with minimal impact on daily number of piles installed and using the same pile installation crew as per normal (non-energy) pile driving, but requires pre-casting of pipes and careful transport to site. Results from the in situ thermal testing and from the long term simulations (under an exemplary given thermal load distribution) are summarised below.

6.1 TRT: Effective thermal conductivity

The various tests were conducted between October 2019 and January 2020. TRT tests were not performed on the connecting straight pipes embedded in the precast pile segments as they did not pass the pressure test after installation (dashed lines in Figure 2-B). This is likely due to leaks in the splicing. Figure 5 shows the measured average fluid temperature, T_{avg} during testing. These temperature response curves are like typical borehole TRTs, with the temperature increasing dramatically for the first few hours and reaching a steady state at a later stage. It is found that the fluid temperature was influenced by the ambient temperature, shown by the fluctuating record seen particularly in TRT1 and TRT2. These were particularly hot days.

In this work, the TRT data were simply interpreted by the application of ILSM and the use of Equation (2) to evaluate thermal conductivity. The actual heat exchanged within the ground, rather than the nominal 1.5 and 3.5 kW, Q , was evaluated as $Q = \dot{m}C_p(T_{\text{inlet}} - T_{\text{outlet}})$, where \dot{m} is the fluid mass flow rate (kg/s), C_p is the specific heat capacity of water taken as a constant value of 4,186 J/kg/K and T_{inlet} and T_{outlet} represent the measured inlet and outlet fluid temperatures (K). It should be noted that the power disruption period in TRT1 was excluded for the thermal conductivity evaluation. The resulting effective thermal conductivities from each TRT and key testing conditions, including the measured Q are summarised in Table 2.

It is interesting to observe that the average actual heat exchange rate of TRT1 and TRT2 was slightly higher than the expected power from the heating elements. This is most likely due to the tests were conducted over hot summer days thus additional heat energy from the ambient environment may be added into the system.

It is found that the effective thermal conductivity of the ground evaluated from these four tests and configurations is relatively close to each other. The average actual heat exchange Q was computed for each test and used in Equation (2) to evaluate the effective thermal conductivities λ_g .

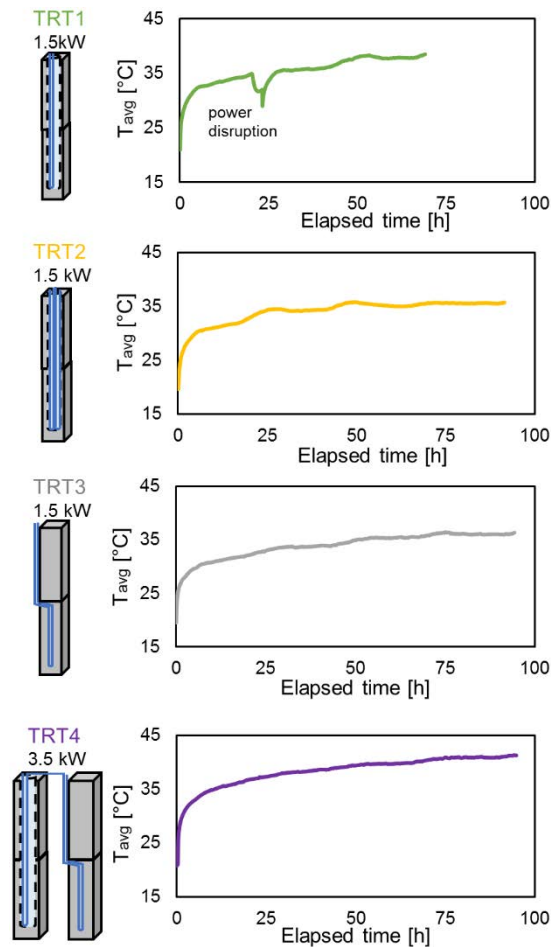


Figure 5. TRTs results expressed in terms of average fluid temperature

Results in Table 2 shows that the highest effective thermal conductivity of the ground is found to be 2.6 W/m·K in TRT2 and the lowest, 2.1 W/m·K in TRT1.

This is higher than the values of thermal conductivity for this geology obtained from small scale laboratory testing (Barry-Macaulay, Bouazza, Singh, Wang, & Ranjith, 2013). This is somehow expected and may be due to the presence of groundwater flow in the project site, which enhances the thermal conductivity of the ground. This condition is not covered in the laboratory tests. There are no other known in situ measurements available to compare.

6.2 Long-term performance simulations

To get an indication of the long-term thermal performance of these energy driven piles, a detailed FEM model was built (as per Section 5) assuming

pile groups in which Pile A type piles are 2 metres apart. Piles of 24 m (two segments) and of 36 m (three segments) were modelled. Figure 6 summarises the model and includes key boundary conditions. Connecting adjacent piles in series increases total pipe length between inlet and outlet of each GHE to render adequate fluid temperature coming in and out of the GSHP in the geothermal plant. An effective λ_g of 2.4 W/(m·K) is used in the models. This was evaluated using the TRT4 data which is considered as the most stable thermal response and the result of also modelling the TRTs, thus it is considered the most reliable value. Note that TRT2, with double U-loops, resulted in a higher λ_g of 2.6 W/(m·K) than the one used in the models.

Long-term thermal performance is highly sensitive to thermal load distribution and operation of the system as described in Section 5. A balanced load, with six months of cooling and six months of heating operating 12 hours day for 50 years is modelled. Peak heating and cooling loads were varied successively to render the design water temperature limits (0-35°C). Figure 7 summarises the results of these analyses. A pair of 36 m deep energy driven piles, which result in a single inlet and outlet with a total GHE pipe length of approximately 275 m, can deliver 2.2 kW peak thermal capacity in heating and in cooling.

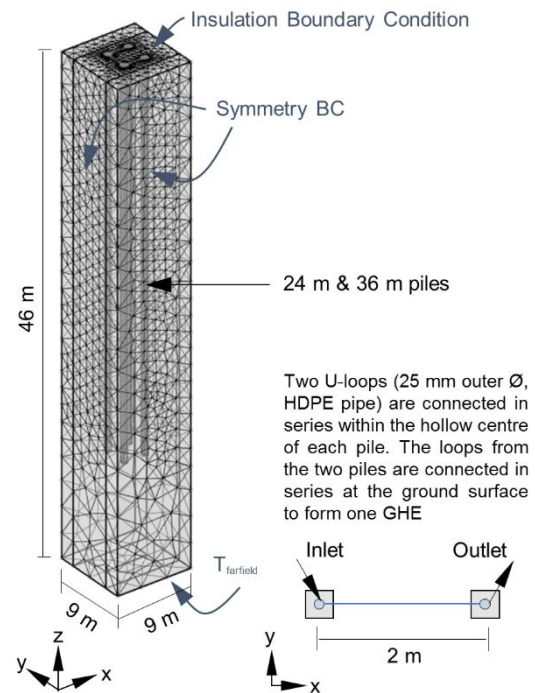


Figure 6. FEM model and key boundary conditions

Table 2: Testing conditions and in-situ TRT results

TRT No.	Heating element (kW)	Average actual heating power (kW)	Flow rate (L/min)	Power/active pile length (W/m)	Gradient m	Effective thermal conductivity (W/m·K)
1	1.5	1.63	28.8	74.1	2.8	2.1
2	1.5	1.58	21.0	70.5	2.2	2.6
3	1.5	1.49	28.2	67.7	2.2	2.5
4	3.5	3.49	21.0	79.3	2.7	2.4

A floating 24 m deep energy driven pile pair, with approximately 180 m of total pipe length, shows a potential thermal yield of 1.5 kW (Figure 7-top); in both cases resulting in a long term annual fluid temperature in the piles within the design limits (Figure 7-bottom).

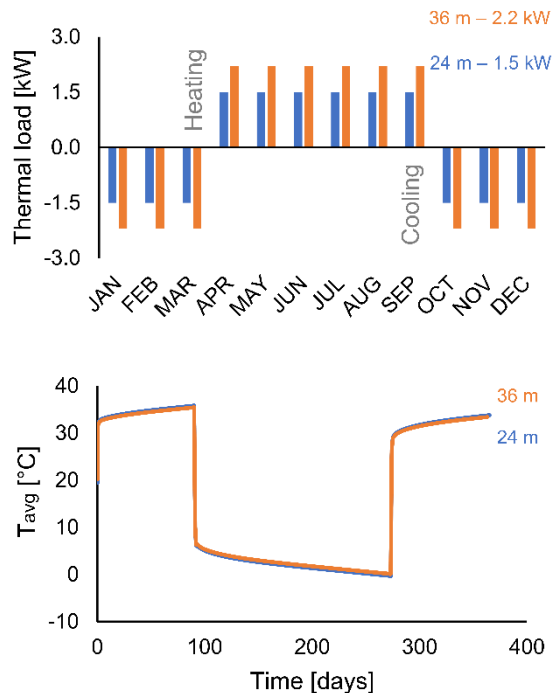


Figure 7. FEM model results showing thermal yield per pair of energy piles (top) and resulting long-term annual fluid temperature (bottom)

At the Fishermans Bend site, this means that all or a large portion of the space heating and cooling demand of a 4 story residential building, for example, could be satisfied with a shallow geothermal system that uses its piled foundations as ground heat exchangers. Any remaining balance can be provided by additional borehole or pushed-in U-loops integrated into the geothermal system or by means of traditional auxiliary HVAC (heating, ventilation, and air conditioning) systems, of much smaller capacity than needed otherwise without geothermal energy. Given the average overall higher efficiency of GSHP systems over traditional HVAC systems, lower GHG emissions are to be expected as well as a cost reduction in providing energy infrastructure to the new district if widely adopted.

7 CONCLUSION

A small-scale field trial was conducted at Fishermans Bend to test constructability and speed of installation of the first energy driven piles in Australia. Three different configurations were trialled and their long-term thermal yield investigated numerically, based on the effective thermal conductivity obtained from various in-situ Thermal Response Tests (TRTs). Connecting pipes embedded into the pre-cast pile segments proves promising but difficult. Embedding pipe in the bottom-most segment and rolling it out on site as the pile segments are driven into the ground proved effective, fast and safe (Pile B type), adding


between 5 and 15 minutes per 36 m deep pile, with low impact on pile production and using the same pile installation crew as per usual installations. Pre-casting of pipes off site is cost effective. Installing geothermal pipes in hollow driven piles (Pile A type) proved to also be effective, fast and safe, with no additional time added to a construction program, but requiring a potentially different crew to add pipes once pile heads on the surface are cut off. In both configurations, the use of piles rather than purposely built boreholes as ground heat exchangers reduces installation costs significantly, estimated in up to 90%. Results from the in-situ TRT show consistent effective ground thermal conductivities (upper ~24 metres), rendering values between 2.1 and 2.6 W/(m·K). Pile A type with double U-loops connected in series in pairs show a long-term energy yield of 2.2 kW peak for a 36 m deep piles and an assumed balanced annual thermal load. This figure is highly dependent on the thermal load distribution designed for and operation the geothermal system, but does indicate the high potential to utilise energy piles on site to satisfy all or part of the heating and cooling required for buildings in Fishermans Bend and thus contribute to a more sustainable future.

8 ACKNOWLEDGEMENTS

Ground Source Systems and Melbourne University's Industry collaboration grant are duly acknowledged.

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SESSION 2
SUSTAINABILITY AND
THE RE-USE OF
FORMER QUARRIES

Keynote Address

Working Towards Net Zero Emissions – Role of Geo-professionals

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ABSTRACT

The global movement in climate change protection is to work towards Net Zero Emissions by around 2050. Net zero emissions refers to reducing greenhouse gas emissions to zero, or as close to zero as possible and offsetting any remaining emissions (e.g. clean energy projects).

This paper outlines some of the steps geo-professionals should take to understand the project Sustainability and Resilience requirements, and areas where we can influence design and construction to meet or exceed those requirements. Sustainability, in relation to geotechnical and geo-environmental work, is an integrated process that balances the social, environmental and financial aspects of planning, design and construction, while managing risk, safety, quality and durability to acceptable standards. Resilience is the ability to cope with uncertain yet extreme events and climate change that may occur over the life cycle of the infrastructure, and to allow expeditious recovery and reconstitution of critical services with minimum impact to public safety and health, the economy, and national security. Geo-professionals are at the forefront of being able to contribute towards sustainable and resilient infrastructure in areas ranging from innovative investigation techniques, use of alternative sustainable resources, reuse of existing foundations, minimising waste, and efficient designs to minimise construction time and materials. Some examples are given in this paper to illustrate where geotechnical designs have contributed to achieving sustainable outcomes.

Keywords: sustainability, resilience, geo-environmental, engineering, infrastructure, design and construction

1 INTRODUCTION

Brundtland (1987) defines sustainable development as development that meets the needs of the present without compromising the ability of future generations to meet their own needs. It is an integrated process that balances the social, environmental and financial aspects of planning, design and construction, while managing risk, safety, quality and durability to acceptable standards. The Infrastructure Sustainability Council of Australia (ISCA) has added governance to social, environmental and financial aspects to form the quadruple bottom line on sustainable developments. One of the elements of governance that has been high on the agenda lately is the avoidance of modern slavery (i.e. ensuring products are not sourced from sweat shops in third world countries).

The author is of the opinion that designing for resilience goes hand in hand with sustainable developments. Resilience is the ability to cope with uncertain yet extreme events and climate change that may occur over the life cycle of the infrastructure, and to allow expeditious recovery and reconstitution of critical services with minimum impact to public safety and health, the economy, and national security.

Developing sustainable infrastructure strategies is increasingly important because in Australia, around 70% of our greenhouse gas emissions are either directly attributable to, or influenced by infrastructure development which is a major area of work for geo-professionals. Such strategies are typically being developed by government and other agencies such as Australian Sustainable Built Environment Council

(ASBEC), ISCA, Green Building Council of Australia and ClimateWorks Australia with the support of large industry partners.

Therefore, the big question is, what role can geo-professionals play in working towards a zero emissions future in sustainable infrastructure development?

Many of us may argue that we have always played a role in sustainable infrastructure development by coming up with cost- and time- effective designs. In some ways this is true – less cost and less time in construction do often equate to the use of less materials and less energy in construction, and may also enhance the social outcome. However, well before the emergence of sustainability awareness in construction in the 1970s, contractors have always demanded engineering design solutions to be cost- and time- effective in order to be price competitive without necessarily thinking about the broader issues of sustainability. The author's opinion is that this is a "business as usual" approach. Sure, this approach does generally contribute to less carbon emission, but it is neither a strategy nor an approach that could help us to achieve Net Zero Emission vision in as short a time as possible.

Fortunately, according to Denis (2019) many large companies in the construction industry are taking a more serious and proactive approach to the climate change protection movement by making Net Zero Emission pledges. A lot of effort is being made in relation to sustainability in the aboveground space (e.g. solar energy, building insulation, LED lighting, internal building materials etc.) but sustainability below the ground has probably lagged behind the

aboveground space and there is an opportunity for geo-professionals to really make big leaps forward in this space through looking at what we normally do through a sustainability lens. Geo-professionals are in an excellent position to assist the infrastructure industry to do the same by adopting a more holistic approach, rather than simply continuing the “business as usual” approach. In order to do this, we need to start adopting the following approach:

- Understand the language and the integrated process on sustainable development;
- Gain an appreciation of the project requirements on sustainability;
- Develop the ability to compare carbon emission outcomes for different designs;
- Have greater interaction with sustainability discipline team members on the project;
- Communicate the benefits of the preferred or selected solutions in relation to sustainability to the stake holders.

2 SUSTAINABILITY DESIGN PROCESS

2.1 Categories of Sustainable Designs

Sustainable design involves the balancing of environmental, economic and social factors, so that the geotechnical system developed satisfies the safety and code requirements and can be constructed within the allotted time frame for a competitive price. Such a design involves efficient scheduling, the use of available local resources, and the avoidance of excess material and labour costs that may be inherent in over-conservative design. Thus, a key aspect of sustainability is minimisation of resource use, costs and environmental impacts. For example, sustainability can be enhanced by the following procedures:

- Use of renewable energies such as wind, solar, hydro power generation and geothermal energy;
- Minimising the use of concrete and other manufactured materials;
- Minimising construction time;
- Minimising waste;
- Minimising energy use;

- Minimising “whole-of-life” cost by considering Net Present Value of both capital and future maintenance cost and resources;
- Maximise the use of site won earthworks materials by modification (if necessary) to meet material strength and durability specifications;
- Retrofitting and reusing existing foundations or geo-structures (where appropriate);
- Minimising intrusive geotechnical investigations by the supplementary use of innovative investigation techniques such as geophysics;
- Designing and constructing reliable and resilient geo-structures to cater for natural and man-made hazards.

The concepts of sustainability and resilience should be considered concurrently. Resilience aims to develop robustness against disturbances, and to enable rapid recovery following such disturbances.

The aim should be to ensure that resilience in geotechnical infrastructure is developed while sustainable practices are undertaken.

2.2 Project Sustainability Requirements

Most major infrastructure projects, particularly those initiated and funded by the government, now usually have project specific sustainability requirements. Unfortunately, these are usually considered by the Contractors and their specialist sustainability consultants without input from the geo-professionals at the project development stage. The author is of the opinion that the geo-professionals should start paying more attention to these project requirements so that we can play an early role in identifying potential opportunities.

In these projects, a number of general sustainability requirements are usually specified, which may include those discussed in Section 2.1 above, and a set of project specific targets. An example of a project specific sustainability targets for a recent major Transport for NSW (TfNSW) infrastructure project in NSW is shown in Table 1.

Table 1: Example of Project Specific Sustainable Targets for a recent TfNSW Infrastructure Project in NSW

No	Target
1	Minimum 5 Star Rating using the Green Building Council of Australia Green Star Design & As Built Rating for the Project.
2	Achieve at least 15% improvement in total annual energy consumption over a reference station based on Section J minimum performance requirements as defined by the National Construction Code Building Code of Australia (NCC BCA) using JV3 methodology. The 15% design improvement and reference station must include all building related energy end uses (excluding process, communication and specialist equipment energy loads).
3	Use a minimum 5% bio diesel mix for all diesel powered plant and equipment and a minimum 10% blended ethanol mix for all petrol powered plant and equipment wherever possible.
4	Offset at least 25% of the total electrical needs of the Contractor's Activities using either or a combination of: (i) purchase of Australian Carbon Offset Credits; and / or (ii) purchase of renewable energy from an Accredited Renewable Energy Supplier renewable energy or carbon offsets.

No	Target
5	Reduce greenhouse gas emissions from the Contractor's Activities by at least 20% from the project baseline greenhouse gas footprint, to be demonstrated determined using the "TfNSW Carbon Estimation and Reporting Tool (CERT)"
6	Recycle or reuse at least 95% of inert and non-hazardous construction and demolition recyclable waste, excluding spoil.
7	Beneficially reuse 100% of reusable spoil.
8	Recycle or reuse 60% of office waste.
9	Use a maximum total construction water demand of [target to be identified in the Sustainability Plan] kL consisting of Mains Water Consumption Target to be identified in the Sustainability Plan] kL of water from potable sources and [Non-Potable Water Consumption Target to be identified in the Sustainability Plan] kL of water from non-potable sources.
10	Achieve a predicted operational potable water demand reduction of 33% below a reference station, to be demonstrated using the Green Star Sydney Metro Tool credit 22 Potable Water Reduction Performance Pathway
11	60% of reinforcing bar and mesh used during construction to be produced through energy reduction processes.
12	Greenhouse gas emissions must be less than the Carbon Emission Target of [target to be identified in the Sustainability Management Plan] tCO ₂ e during the Contractor's Activities.
13	A maximum consumption of [Electricity Consumption Target to be identified in the Sustainability Management Plan] kWh of electricity during the Contractor's Activities.
14	A maximum consumption of [Electricity Consumption Target and Fuel Consumption Target to be identified in the Sustainability Management Plan] kWh of electricity and KL of fuel during the Contractor's Activities.
15	A minimum 15% reduction in the environmental footprint of the materials used for the Contractor's Activities, compared to a business-as-usual case.
16	Reduce Portland cement content in concrete by an average of at least 25% through replacement by supplementary cementitious materials such as fly ash or slag. Opportunities for further reduction will be investigated by carrying out trial mixes and testing.

It can be seen from the table above that geo-professionals can have direct influence on Items 5, 6, 7, 12, 13, 14, 15, and 16 by considering alternative designs that could benefit the sustainability outcome during the Contractor's activities. Therefore, it is essential that collaboration and integration of design and construction activities from the geo-environmental engineering and other disciplines such as construction and sustainability specialists take place in order to maximise the sustainability outcomes for each project, and to ensure the sustainability outcomes achieved by the geotechnical design are recognised by the sustainability specialists.

2.3 Holistic Process

Basu et al (2013) consider sustainability can be looked upon as a dynamic equilibrium between four E's - engineering design, economy, environment and equity (same as the "social" aspect used in this paper), as described in Figure 1.

Basu et al (2013) describes that "the four E's approach of sustainable engineering, the sustainability objectives that may be incorporated in geotechnical projects are: (i) involving all the stakeholders at the planning stage of the project so that a consensus is reached on the sustainability goals of the project (such as reduction in pollution, use of environment friendly alternative materials, etc.), (ii) reliable and resilient design and construction that involves minimal financial burden and inconvenience to all the stakeholders, (iii) minimal use of resources and energy in planning, design, construction and maintenance of geotechnical facilities, (iv) use of materials and methods that cause minimal negative

impact on the ecology and environment, and (v) as much reuse of existing geotechnical facilities as possible to minimise waste. This approach aims at reaching a dynamic equilibrium between engineering integrity, economic efficiency, environmental effectiveness, and social acceptability and equity."

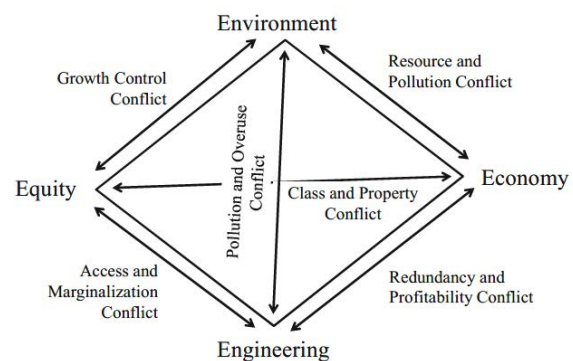


Figure 1. The four E's of sustainability in engineering projects (Basu et al, 2013)

Misra and Basu (2011, 2012) developed a sustainability assessment framework for pile foundation projects. The framework considers a life-cycle view of the pile construction process and a multicriteria framework of geotechnical design as shown in Figure 2.

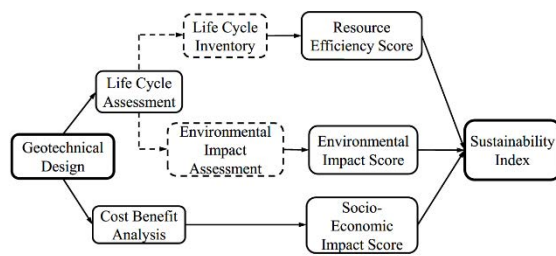


Figure 2. Multicriteria Framework of Pile Foundation Design Proposed by Misra and Basu et al (2011, 2012)

The multicriteria framework of pile foundation design shown in Figure 2 could equally be applied to other geo-environmental projects, and or material production processes.

An essential element of the multicriteria assessment process is the weighting of each criteria in relative importance (Belton and Stewart, 2001) and this must be carried out with all project stakeholders on a project specific basis.

For example, Holm et al (2013) presents a case study on the use of multicriteria framework for the disposal of dredged, contaminated material at the Port of Gothenburg in Sweden. A number of disposal methods including landfill, sea disposal, rock chamber disposal and solidification / stabilisation options were considered. Each option was assessed in relation to energy use and climate impact both of which include the contributory components in production, transport, construction and maintenance. Weighted scores were then given to the environment, social and economic targets and summed to form a total performance score. In that example, economic and environmental criteria were given 2 and 1.5 times the weight of social criteria, respectively, and it was demonstrated through the multicriteria analyses that the rock chamber disposal and solidification / stabilisation options provided the highest performance score. It would appear that various organisations have established in-house multi-criteria analysis to assess sustainability of options in the remediation space, and there is no reason why this approach cannot be used in the geotechnical space.

Another important aspect of sustainability is innovations that are quite often associated with such challenging projects. Innovations are important to sustainable development because they can contribute to social, economic and financial benefits.

2.4 Embodied Carbon/Emission Estimates

The term embodied carbon is the sum of all the greenhouse gas emissions attributed to the materials throughout their life cycle, from extracting from the ground, manufacturing, construction, maintenance and end of life/disposal. It is also called the carbon footprint and is measured by the sum of all greenhouse gases (GHGs) to produce the materials. In order for comparison to be made, an equivalent carbon dioxide emission factor, usually in units of

kgCO₂e per m³ or per tonne, is used for different materials.

The CO₂ emission for different materials will differ from country to country due to different resources available and will also be dependent on the site location with respect to transportation requirements. For concrete, it will also depend on the mix design of the raw materials used for different concrete strengths (Kim et al, 2016, Purnell, 2013). Furthermore, other factors such as CO₂ emission associated with long-term operation of the constructed product may also influence the overall sustainability score of the project. For example, NRMCA (2008) quoted research conducted by the National Resources Council of Canada has shown that heavy trucks traveling on concrete pavement accumulate statistically significant fuel savings, ranging from 0.8% to 6.9%, versus asphalt pavement for major urban arterial highways. These fuel savings lead to reductions in greenhouse gas emissions and air pollutants, with equivalent CO₂ reduction ranging from 1,039 tonnes to 8,950 tonnes over the life of the pavement.

On major infrastructure projects, the estimate of CO₂ emission and/or reduction from a reference design is usually carried out by the project sustainability specialist. However, the contribution from the geo-environmental profession can be over-looked unless there is close collaboration between the different disciplines. For this to take place, it is useful for the geo-professionals to have some awareness of the CO₂ emission factors commonly used for different materials or processes. For example, TfNSW has provided extensive policy and guidelines for sustainability design and construction, including a Carbon Emission Reporting Tool (CERT) v2 (2017) which can be downloaded from the website provided in the reference section of this paper. Some of the CO₂ emission factors used in CERT v2 are summarised in Table 2.

Table 2: Extract of CO₂ Emission Factors used in The Carbon Emission Reporting Tool (CERT) v2 provided by Transport for NSW

Material / Process	Unit	Emission Factor
Ready mix concrete:		
≤ 20 MPa	kg.CO ₂ e/m ³	257
25 MPa	kg.CO ₂ e/m ³	285
32 MPa	kg.CO ₂ e/m ³	332
40 MPa	kg.CO ₂ e/m ³	398
50 MPa	kg.CO ₂ e/m ³	445
60 MPa	kg.CO ₂ e/m ³	445
80 MPa	kg.CO ₂ e/m ³	539
> 80 MPa	kg.CO ₂ e/m ³	633
Portland Cement	kg.CO ₂ e/t	948
Fine Aggregates	kg.CO ₂ e/t	4.3
Coarse Aggregates	kg.CO ₂ e/t	5.7
Recycled (coarse) aggregates	kg.CO ₂ e/t	4.4
Reinforcement steel bars (Australian product)	kg.CO ₂ e/t	1900

Material / Process	Unit	Emission Factor
Reinforcement steel bars (Imported)	kg.CO ₂ e/t	2000
Reinforcement steel mesh (Australian product)	kg.CO ₂ e/t	2600
Reinforcement steel mesh (Imported)	kg.CO ₂ e/t	2700
Asphalt	kg.CO ₂ e/t	65
Reinforced concrete pipes	kg.CO ₂ e/t	280
Steel pipe and tube (Australian product)	kg.CO ₂ e/t	3000
Steel pipe and tube (Imported)	kg.CO ₂ e/t	3100
HDPE pipes	kg.CO ₂ e/t	3025
PVC pipes	kg.CO ₂ e/t	3569
Timber (structural softwood)	kg.CO ₂ e/t	-660
Timber (structural hardwood)	kg.CO ₂ e/t	-737
Timber, MDF/Particleboard	kg.CO ₂ e/t	-175
Timber, plywood	kg.CO ₂ e/t	-335
Waste Disposal to landfill:		
Transport (50 km)	kg.CO ₂ e/t	6.45
Transport to recycling centre (22 km)	kg.CO ₂ e/t	2.838
Construction and demolition waste (inert)	kg.CO ₂ e/t	0
Construction and demolition waste (timber and vegetation)	kg.CO ₂ e/t	600
Construction and demolition waste (mixed waste)	kg.CO ₂ e/t	200
Transport Emission:		
Rigid truck	kg.CO ₂ e/tkm	0.129
Articulated truck	kg.CO ₂ e/tkm	0.0726
Rail	kg.CO ₂ e/tkm	0.0255
Shipping	kg.CO ₂ e/tkm	0.00893

3 PROJECT EXAMPLES

Presented below are some examples of engineering projects in which the author considers sustainability outcomes have been achieved. For many of the projects, no specific focus was made on sustainability at the time. Nevertheless, they illustrate that if the requirement to balance the financial, social, and environmental factors are considered, sustainable solutions are often achieved, and that potentially greater success can be achieved in developing sustainable geotechnical solutions if there is more collaboration between the geotechnical engineers and other disciplines, including sustainability specialists.

3.1 Ballina Bypass Alliance

The Pacific Highway Upgrade program was initiated by the NSW Government in the 1990's when there were divided dual carriage ways in each direction for

less than 30 per cent of its 700km length. There were many accident blackspots along the route with relatively high rate of fatalities. For example, there were 44 fatal crashes on the highway in 2003, most of which occurred on undivided sections of the highway. It took the state and federal governments considerable will power to allocate joint funding to upgrade the highway to divided dual carriage way in each direction between Hexham in NSW and the QLD border that would see a construction program costing over \$10 billion with completion expected around 2021.

In relation to sustainability, the Pacific Highway Upgrade program demonstrates that it is not just money that matters. The upgrade would have saved many lives over the years by reducing accident blackspots and fatal accidents, and cutting considerable travel time for the general public and reduced commercial road-freight costs.

An example the author would like to use is the Ballina Bypass Alliance (BBA) Project. The bypass is a 12km long four-way dual carriageway linking the Pacific Highway in North Ballina with the Bruxner Highway Intersection in South Ballina. It extends from South Ballina to North Tintenbar in NSW, and more than half of the route is underlain by some of the most compressible soils in Australia, and to depths exceeding 25 m.

Previous geotechnical studies on the project had indicated that conventional staged embankment construction to maintain stability and surcharging to limit post-construction settlement to acceptable limits would have required a construction period of 13 years due to the presence of 25 m deep very soft clays at the southern bridge approach to Emigrant Creek North. The author considers some of the following examples of innovations on the project have contributed to sustainable infrastructure development:

Alliance Delivery – The BBA was the first of the Pacific Highway Upgrade program to adopt the alliance delivery model to solve the technical complexity of the soft soil engineering on the project. An alliance delivery model was likely to be more costly than the traditional Design and Construct (D&C) or Design Construct and Maintain (DCM) models, but was chosen because it was considered more likely to develop innovative solutions and avoid costly claims associated with traditional D&C or DCM models. Through close collaboration with the road authority which included capital works and maintenance divisions, contractor and design consultants, the project was completed in a 4.5 year period between 2008 and 2012 (much shorter than the original estimate of 13 years). The total project cost was \$640 million. A reduced construction time means lives were saved due to reduction in fatal crashes as well as providing better road user benefits by reductions in travel time (i.e. social benefits). To manage post-construction settlement (see Low Embankment Strategy discussed below), dialogue with the maintenance division of the road authority was engaged, and a \$10 million maintenance budget was allocated specifically for this project. To the author's

understanding, this was the first time where a dedicated maintenance budget was made for a section of the highway in addition to the usual budget for maintaining all roads statewide. In order to manage public expectations regarding the need for short- and long-term maintenance, a public education program including commercially made videos of soft soil behaviour and ground treatment methods, was implemented. The videos were displayed at the community display centre during the works. This was a first time use of such public education program on the highway upgrade program.

Low Embankment Strategy – over 6 km sections of the route away from bridge approaches were underlain by deep soft soils. The initial cost estimate to treat these sections using conventional soft ground treatment to enable plain concrete pavement to be adopted was in the vicinity of \$50 million. If the project budget was increased by this amount, it would have meant that subsequent sections of the highway would have had to be delayed until further funding was available from the state and federal governments. This was considered unacceptable from a social perspective in relation to the urgent priority of reducing accident blackspots and saving lives. Therefore, instead of conventional ground treatment to enable rigid pavement to be adopted, a low embankment strategy was devised based on a whole-of-life approach. It involved keeping the embankment as low as possible, while maintaining flood immunity for at least one traffic lane in a 1:20 year flood event. To allow for post-construction settlement, a staged, flexible pavement approach, using moisture-resistant pavement material and reshaping after one year of operation, was adopted together with a dedicated maintenance budget allocated for the project as discussed above. The fill was placed as early as possible together with a flood gap strategy which comprised a large bull-dozer purchased and dedicated to the project to ensure parts of the embankment could be removed quickly to allow flood water to pass.

Vacuum Consolidation (VC) – amongst other areas of innovation on this project, the BBA project became the first site in Australia to adopt the vacuum consolidation method (Kelly and Wong, 2009) in reducing post-construction settlement. VC enabled the initial 8 m of filling including surcharge to be placed rapidly without instability issues, followed by a further 6 m without VC to consolidate the soft soils. In total, 6.5 m of settlement was induced from the VC and surcharge at this site. The conventional staged construction to allow strength gain to maintain embankment stability would have delayed the project by many years, and the use of rigid inclusion techniques would be expensive due to the depth of soft soils, and less sustainable from the point of view of concrete usage.

3.2 Woolgoolga to Ballina Pacific Highway Upgrade

Woolgoolga to Ballina is the final link in the Pacific Highway, between Hexham in NSW and the Queensland border, to be upgraded to four lanes to improve traffic safety, reduce travel time and improve

amenities to local communities. It is the last 155 km section of the highway upgrade project, with a budget of \$4.9 billion to be completed around 2021. Similar to the BBA project described in Section 3.1, there are many social, financial and environmental factors to be considered in such a major infrastructure project.

Approximately 26 km of the route is underlain by soft soils of varying thicknesses and properties. In the author's opinion, the geotechnical engineering associated with the treatment of the soft ground for the embankment and road construction has definitely contributed significantly to a sustainable solution for this project. Many years before construction took place, the NSW road authority had a desire to de-risk the soft soil issues associated with this project from the lessons learnt from the BBA project, and to reduce the cost associated with the need to use hard treatments which are less sustainable due to the high demand for natural materials such as high quality gravel for construction of stone columns, and cement and aggregate for concrete injected columns. The planning led to a study of potential Early Works by preloading and surcharging critical soft soil sites well in advance of the major works in 2014, using interim funding available at the time. The planning study is described in Wong (2012) although the project name was not mentioned at the time due to confidentiality reasons. The Early Works study framework is illustrated in Figure 3, and involved looking at different scenarios of time available for Early Works to target where conventional surcharge may be utilised to achieve post-construction settlement and differential settlement criteria.

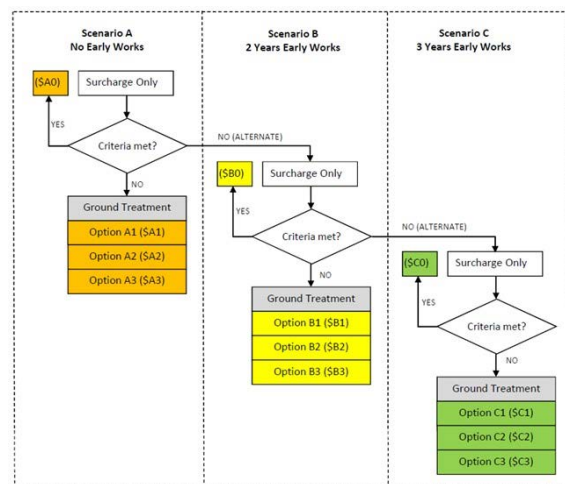


Figure 3. Flowchart showing Framework for Early Works Consideration (Wong, 2012)

A total of 22 sites was considered initially. During the actual project works, the number of Early Works treatment sites were reduced to about half the numbers using a multi-criteria selection process considering land acquisition at the time, material availability and post-construction settlement risk. The Early Works for the selected sites were eventually implemented in 2013 by a number of subcontracts prior to the major works in 2014 via a Delivery Partner model.

By planning and conducting the Early Works on this project, the following sustainable benefits were derived:

- Savings in the use of high emission resources such as cement and concrete used in other hard treatment options.
- Lower cost by adopting conventional earthworks rather than hard treatment, which also amounts to less CO₂ emission;
- Lower risk of unexpected post-construction settlement because response actions were able to be implemented from longer periods of settlement monitoring available prior to pavement construction;
- By reducing the risk of unacceptable post-construction settlement, maintenance cost in the long-term is reduced which also amounts to less CO₂ emission.

3.3 Barangaroo South Pile Foundations

The 3 high-rise commercial towers at Barangaroo South range from 39 to 49 storeys, providing 490,000 m² of gross area. It was designed to become one of Australia's first large carbon dioxide-neutral developments. Importantly, all three towers offer 6 Star Green ratings.

The core of each of the three towers is supported on groups of piles ranging from 1 m to 1.5 m in diameter. The outer columns have spacings of up to 10.5 m and carry design working loads and ultimate loads that are in excess of 80 MN and 100 MN. These outer columns are supported on 2.4 m diameter piles. Including the

podium structure, there are nearly 1,000 piles to be constructed in a difficult subsurface profile as shown in Figure 4.

The challenges associated with the foundation design for this project include the following:

- The project structural engineer desired a tight settlement criterion of 0.3% of the pile diameter at the pile head to limit distortions of the building façade;
- Rock level ranges from RL+1m to RL-28m, and is overlain by alluvium and filling. Rock levels can vary significantly over short distances and there are potential for buried cliff ledges;
- With about 1000 piles to be constructed in difficult ground (saturated fill and alluvium that are prone to collapse, and the high strength rock is hard to drill); piling represented a major cost and time challenge to the project.

Due to the heavy building loads and stringent settlement criterion, all foundation piles are socketed into the Hawkesbury Sandstone bedrock.

The traditional approach of designing piles in Sydney rock has been to use conventional "Serviceability" bearing pressures that correspond to a settlement within 1% of the pile diameter in accordance with Pells et al (1998) which generally provides economical designs. However, due to the much stricter settlement criterion of 0.3% of the pile diameter on this project, an initial design using factored down serviceability bearing values resulted in long rock socket lengths and costing well above the project budget.

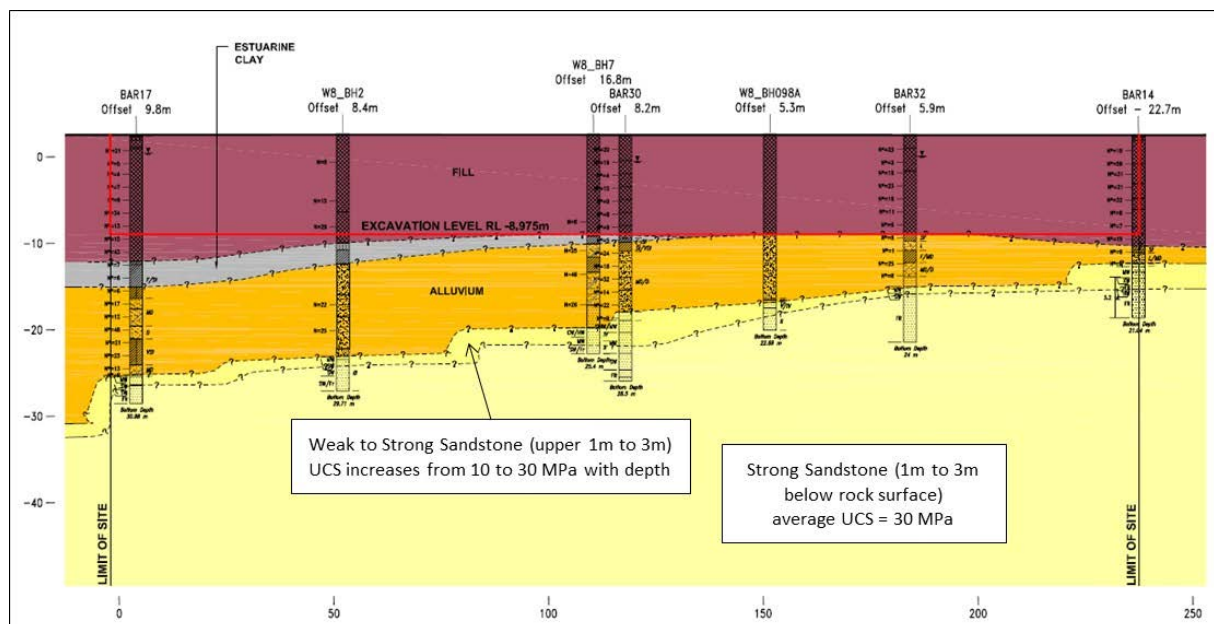


Figure 4. Typical Subsurface Profile at Barangaroo South Commercial Tower Site

A different design approach was therefore adopted using the following strategy:

- Because of high rock strength, strength limit state was not a governing factor for design, enabling the designer to focus on a performance based design;
- Following discussions with the structural engineer, and on the basis that elastic shortening of the pile shaft is relatively small, the design settlement target was changed to 0.3% of pile diameter at the pile base;
- The design was carried out using presumptive rock stiffness values for the various sandstone classes given in Pells et al (1998), and later checked using the results of pile load testing and back-analysis discussed below;
- Two bi-directional O-Cell pile load tests were carried out on prototype 750mm diameter piles (Figure 5), which enabled testing to about 70% of the ultimate base capacity and 100% of the shaft capacity. This enabled the strength limit state to be confirmed and pile stiffness to be back-analysed for checking with building performance requirements;
- A program of proof coring (about 25% of piles within the tower core, and each of the 2.4m diameter tower piles) to confirm the rock profile and assess the potential for cliff ledges;
- Full-time geotechnical presence during piling to confirm founding levels and cleanliness of the pile base, and the use of appropriate drilling tools to achieve a minimum rock socket roughness of 4mm at maximum 200mm spacing.

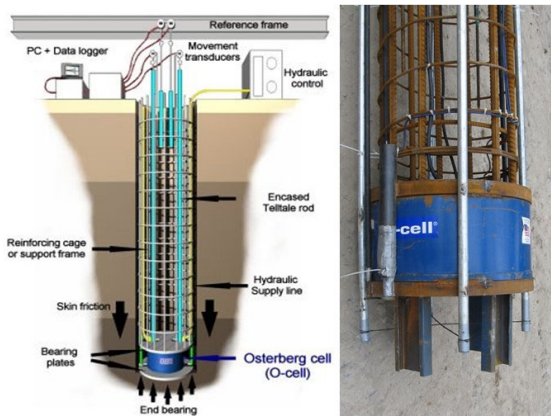


Figure 5. O-Cell Pile Load Testing Arrangement

Descriptions of the O-Cell pile load testing, interpretation and back-analysis results, including non-linear behaviour with load level can be found in Wong and Oliveira (2012) and Wong (2013).

The advantage of utilising this performance based design approach as compared to using a presumptive settlement of 1% of the pile diameter based on published serviceability design values is illustrated in Figure 6 for a 1.8 m diameter pile socketed 6 m into rock (1m in Class V Sandstone, 2m in Class IV Sandstone, and 3m in Class III Sandstone).

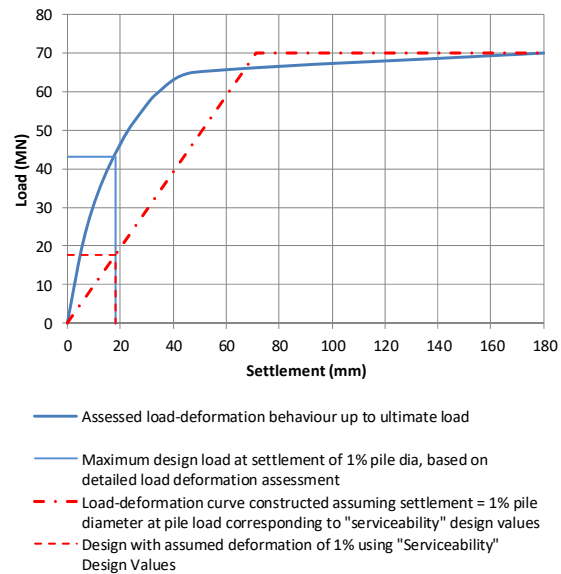


Figure 6. Load-deformation Curves for 1.8 m Dia. Pile with 6 m Rock Socket in Sandstone

It can be seen from Figure 6 that at the same settlement of 18 mm (i.e. 1% of the 1.8 m pile), the actual load based on a proper analysis using appropriate rock properties is over twice the capacity using the simple presumptive approach.

As with most infrastructure projects, the need to optimise the pile foundation design and construction for the Barangaroo project was driven primarily by cost and time. However, in relation to sustainability, we can now reflect on the reduction in CO₂ emission achieved, as described below. This represents a very conservative estimate because only 323 of tower piles (out of nearly 1000 piles in total) are considered, ignoring the podium piles.

- Saving in total rock socket length ~ 40%
- Savings in total pile length ~ 20%
- Total pile length saved ~ 1,200 m
- Total volume of 50 MPa concrete saved ~ 6,264 m³
- Total steel reinforcement saved ~ 970 tonnes
- Reduction in CO₂ emission ~ 4,635 tonnes

The reduction in CO₂ emission shown above is simply from the savings in concrete and steel, and does not include energy savings associated with not having to drill the additional rock sockets and transportation of materials to site. The savings in transport of materials to site also have an indirect benefit to road users and social well-being of the community due to lesser disturbance from reduced heavy truck movements.

3.4 Barangaroo South Piling Platform

Another example of sustainable design by the geotechnical engineers on the Barangaroo South commercial tower project discussed in Section 3.3 is the use of innovative geotechnical analysis and testing method to eliminate the piling platform for construction of the diaphragm walls and those

foundation piles constructed from existing ground level.

Numerous heavy tracked machines were used for the construction of the diaphragm walls for the basement excavation and large diameter piles for the tower foundations. A 600 mm thick working platform constructed using coarse granular aggregates was initially designed for an area of 2 ha due to concerns regarding the uniformity of the existing filled ground. The import of materials, construction, and subsequent excavation and disposal of the materials would have cost an estimated \$1 million. The associated CO₂ emission including material transport both ways, construction and disposal would have been approximately 400 tonnes.

Although it was considered likely that a piling platform may not be required because of the granular nature of the existing reclaimed filling at the site, the challenge was to demonstrate the existing ground could withstand an ultimate bearing pressure of up to 437 kPa for a track width of up to 1 m. The solution, as reported in Wong and Moyes (2014) was to use small strain theory with proof rolling at a machine track operating pressure of only 160 kPa, coupled with a program of plate bearing tests to maximum test pressures of 450 kPa to 550 kPa, Benkelman Beam deflection testing, and pre- and post- ground level surveys following the proof-rolling.

The use of the small-strain theory enabled the secant modulus, E_s and tangent modulus, E_t of the ground under the track loading at different pressures to be interpreted from the testing as shown in Figure 7. A load deformation diagram was then able to be produced and tolerable deformation limits set for the proof loading.

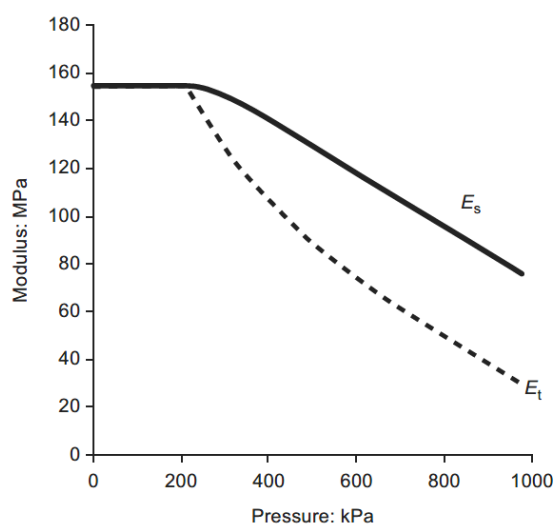


Figure 7. Interpreted average secant modulus, E_s and tangent modulus, E_t from test results using small-strain theory reported in Wong and Moyes (2014)

The innovative methodology adopted successfully eliminated the piling platform for the diaphragm wall pile foundation construction at existing ground level, and identified a soft area which was associated with a

poorly backfilled service trench, and enabled this area to be bridged-over using steel plates.

Although the CO₂ emission reduction associated with elimination of the piling platform is not huge (~ 400 tonnes), the cost and time saving, together with other indirect social benefits of reducing heavy vehicle traffic on local roads, noise and vibration reductions certainly made it worthwhile. The learnings from the innovative use of small strain theory also provide benefits with respect to potential adaptation of the technique in future projects.

3.5 Waikato Expressway – Hamilton Section

The Hamilton Section of the Waikato Expressway is a 21.8 km highway around the eastern fringe of Hamilton City in New Zealand. The project is being delivered as an Alliance comprising the project owner, New Zealand Transport Authority (NZTA), contractors Fletcher and Higgins, and designers Beca and Coffey, and is due for completion in around 2021.

Because of the requirement to design for relatively high seismic, low probability (i.e. high return period) earthquake events, a more sustainable, resilient design approach was adopted by the Alliance. As usual, NZTA sets project Minimum Requirements (MR) for each project. The alliance delivery mechanism provided an opportunity to explore potential solutions that cannot strictly meet the MR at a reasonable cost, but one that is considered to have an acceptable risk due to low probability of occurrence and consequence of damage in an adverse earthquake event. In any case, the solution must be sufficiently resilient to enable temporary repairs to be made quickly to enable emergency and essential traffic usage of the road following such an event, and the consequence of damage must not have adverse impact to the environment. A high-level illustration of the design process is shown in Figure 8 below.

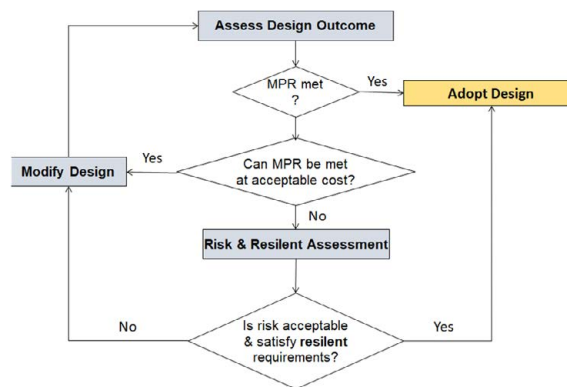


Figure 8. Illustration of Design Process for Resilience

An example of the application of this design process was reported in Wong et. al. (2019) for the northern embankment over the Mangaharakeke gully near the southern end of the project.

The site was underlain by liquefiable soils extending down to about 7.5 m depth. The initial concept design that met the MR was a lattice structure constructed

using continuous flight auger (CFA) concrete columns, but this solution was considered by the Contractor to be too expensive and an alternative solution was needed.

The adopted solution was Dynamic Replacement (DR) utilising Greywacke rock which was readily available from local quarries. Even with pre-excavation below the ground surface to 1.5 m, validation testing indicated the DR columns could not reach the full depth of the liquefiable soils. However, risk assessment conducted indicated that the risk was acceptable and the as-constructed ground improvement using DR provided adequate resilience which met the following requirements at SLS and ULS earthquake accelerations of 0.09g and 0.24g, respectively:

- FOS (for embankment stability) ≥ 1.5 under static long-term condition
- FOS ≥ 1.0 for 100% SLS earthquake inertia and pre-earthquake soil strength
- FOS ≥ 1.1 ULS with no earthquake inertia and post-liquefied/cyclic softened soil strength
- FOS ≥ 1.0 with 65% SLS earthquake inertia and post-earthquake soil strength
- FOS < 1 is acceptable for ULS with earthquake inertia, but allowable lateral deformation ≤ 250 mm at critical acceleration for one ULS design earth-quake event.

The benefit of using granular DR columns to reduce the liquefaction potential is illustrated by the study conducted for stone columns by Thevanayagam et al, (2006) who presented the improvement in the normalised and corrected Standard Penetration Resistance $(N_1)_{60cs}$ for soils of different permeability, as reproduced in Figure 9. Essentially, an increase in $(N_1)_{60cs}$ represents an increase in liquefaction resistance of the treated soil.

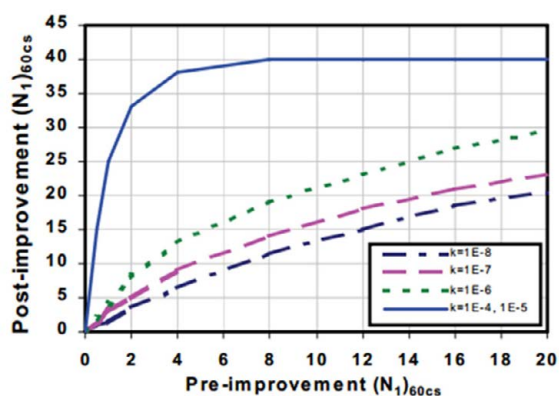


Figure 9. Pre and post improvement $(N_1)_{60cs}$ for stone column replacement ratio of 22.5% presented by Thevanayagam et al (2006)

Figure 9 was used for the concept design of DR for the project, followed by a more detailed design process. Following site trials, the adopted solution comprised 3.6 m diameter columns at 5.8 m centres on a triangular grid to give a design area replacement ratio of 0.35. A more detailed description of the design process and validation testing can be found in Wong et. al. (2019).

Because of the locally available rock for the DR, as compared to the high cost of CFA and high embedded carbon of concrete, the adopted DR solution is considered to be far more sustainable compared to the initial CFA lattice ground improvement.

3.6 One Man's Striving on Sustainability – The 5x4 Hayes Lane Project

The last example given in this paper is from one of the episodes of the TV series "Grand Designs Australia" featured on the Australian Broad Casting (ABC) channel recently (Series 6, Episode 4).

The author has no involvement on this project whatsoever, but was awed and inspired by the owner's focus and strive on maximising sustainability in the design and construction of his home in East Melbourne. The owner builder, Mr. Ralph Alphonso, constructed his 12 m high, 4 storey inner city home on a tiny 5 m x 4 m footprint. His passion in achieving minimum CO₂ emission sees him close to eliminating steel altogether from the structure and building almost entirely from sustainable Australian hardwood, including timber floors, timber walls, timber ceilings, timber structural beams, timber cabinetry, even an engineered timber stairwell that allowed him to go four stories high. The building is designed, built and powered with both passive and active eco-driven processes, materials and performance considerations.

Above all, the author is most impressed by the fact that before the building work started, Mr. Alphonso sank a 65 metre deep geothermal drill hole that will heat and cool his home using heat pump technology. The geothermal system added \$30,000 to the cost of the project, so it is by no means an inexpensive solution, but it was a priority in Mr. Alphonso's vision for his eco-friendly house. "Eventually, the hope is that it will pay for itself" says Mr. Alphonso on the TV show.

A subsequent paper by Crawford (2018), who studied the energy use of Mr. Alphonso's project over a 3 year period, found that "the building has resulted in net operational GHG emissions of 13.8 kg CO₂-e per day (assuming the use of fossil fuel-based electricity imported from the grid)" as compared with an average emission of 18.1 kg CO₂-e per day in Melbourne. Crawford (2018) further concludes that "there is still room for improvement. Data from individual circuits showed that the geothermal heat pump is responsible for the greatest share of energy use. Improvements to the efficiency of the heat pump system and the pumps used for pumping water through the geothermal pipes are thus two areas where efforts to further reduce energy use within the building should be targeted."

Mr. Alphonso's striving for minimising the carbon footprint in the design and construction of his home is an inspiration for all geo-professionals.

The area of geothermal energy is one that has been explored extensively but not yet adopted widely. Traditionally, geothermal energy use has been limited to tectonically active regions. For example, electricity was first generated using steam from a geyser in Italy

in 1904, and New Zealand started commercial power generation using separated steam in 1958 (Johnston et. al., 2011). In the last 20 to 30 years, use of relatively shallow geothermal energy in non-technic active regions including Australia is starting to take place for heating and cooling of buildings thanks to the invention of the heat pump by Lord Kelvin in 1856.

Johnston et. al. (2011) provided a good summary of the emerging technologies of Enhanced Geothermal Systems (EGS) and Ground Source Heat Pumps (GSHPs) which are important because of their potential for widespread use. A simplified summary of the various forms of geothermal systems illustrated by Johnston et. al (2011) is reproduced as Figure 10 in this paper.

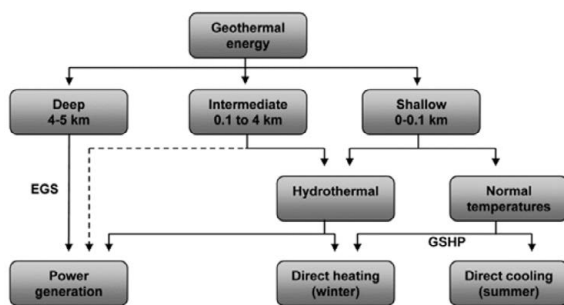


Figure 10. Simplified Summary of the Various Forms of Geothermal Energy by Johnston et. al. (2011) - The dashed line indicates that ground temperatures at intermediate depths (where there are no hydrothermal resources) are not typically high enough for the commercial generation of power using current technology.

Johnston et. al. concluded that “EGS and direct geothermal heating and cooling using GSHPs are emerging technologies that have the potential to significantly reduce the world’s dependence on carbon based energy sources” and “Commercial application of EGS technology is some years off but its enormous potential demands investment for further research. GSHP heat pump technology is available now – policy makers need to be educated and the potential of GSHPs demonstrated.”

Professor Johnston (2020) indicates that the main hindrance in widespread use of GSHP technology in Australia is capital cost of installation and the lack of Government initiatives to subsidise its use as an alternative renewable energy to reduce supply demand, even though it does not generate energy for the grid. Furthermore, Professor Johnston indicates that owners/designers/installers of GSHP systems need to think more smartly about the system requirement according to the temperature range of the region to enable more affordable systems to be installed. For example, he quotes that 90% of the heating in Melbourne is required when the temperature is between 14°C. and 20°C. Therefore, over-design (i.e. installing a system that can provide heating at 3°C temperature) can be avoided by installing a parallel heating system using conventional power that would only be operated for 10% of the time or less.

With respect to infrastructure development, there are good opportunities to use GSHP systems at reduced capital cost via embedment of closed loop systems in in-ground structures such as retaining walls, piled foundations, basement floors and transportation tunnels. Such systems could be used for heating and cooling of transport control centres, metro over-station developments, airport terminals or even potentially extending its use to integrated residential and commercial developments along transportation corridors. Ongoing research and development of geothermal energies will no doubt lower the cost of GSHP systems with time, and hopefully see the use of EGS in generating energy on a viable commercial scale in the near future. It is pleasing to see there are many recent published research works that can be found in publications such as Ferrari and Laloui (2018) on energy piles and other forms of direct geothermal use.

Mr. Alphonso sets a perfect example of what one man can achieve with his passion for a sustainable home development in East Melbourne. Imagine what an infrastructure community can achieve with the drive of geo-professionals, architects and other engineering professionals, and with Government support.

4 CONCLUSION

Geo-professionals have always contributed to sustainable infrastructure development, even if subconsciously, via the need to arrive at time- and cost- efficient designs driven largely by contractors in a highly competitive market. The author is of the opinion that this is a business as usual approach, and considers that more sustainable solutions can be achieved if more conscious efforts are made by using a holistic approach. This requires understanding the different competing factors including government policies, time, cost, social and environmental factors. Major infrastructure projects usually have specific sustainability targets that the geo-professionals should make an effort to understand, and work collaboratively with the client and other engineering disciplines to explore opportunities, including the use of multicriteria selection processes where applicable. Innovation forms an important part of developing sustainable solutions and the use of resilient design principles in a whole-of-life design approach should be considered in major infrastructure projects.

Mr. Alphonso’s 5x4 Hayes Lane Project, whereby he invested in a geothermal heating and cooling system in his home project, illustrates that capital cost is not everything. Energy savings will eventually recover the capital cost in time, in addition to being eco-friendly. Geothermal energy is one of many areas where geo-professionals can potentially exploit in integrated transportation infrastructure projects for the heating and cooling of residential and commercial developments by embedding geothermal closed loop systems within in-ground structures.

5 ACKNOWLEDGEMENTS

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A settlement hazard risk management framework for the development of backfilled open-cut quarries

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ABSTRACT

Surface mining operations typically lead to the disturbance of large areas of land over the life cycle of mines. With growing pressures on land use, particularly in urban areas, brownfield sites such as closed open-cut mines are being considered more and more for redevelopment opportunities. Naturally, these areas require reinstatement to some workable final surface level which is then suitable for the future development. Nevertheless, the business case that supports the decision to develop these complex areas should aim at balancing and optimising the investment required in the rehabilitation of the original site itself, as well as the cost of subsequent infra- and superstructure development. Controlled backfilling of these disused operations is often complex, including considerations of variability and engineering characteristics of the available backfill materials and the influence of groundwater recovery. Variations in the properties of the backfill material may not have any significant impact on some after-use activities (e.g. when the area is intended for conservation or agricultural land use after rehabilitation), but subsequent surface infrastructure development could be severely affected by settlement of backfilled land. The strategic planning and closure process for open-cut mines typically involves a number of different stages and a detailed consideration of the potential behaviour of the backfill is crucial in ensuring successful development. The purpose of this paper is to present a Risk Management Framework (RMF) focused on addressing excessive post-construction settlement. This RMF needs to form part of a more comprehensive geotechnical, and ultimately overall risk management and mine closure plan.

Keywords: settlement hazard, open-cut quarry, risk management framework

1 INTRODUCTION

All surface mining methods involve the extraction or stripping of near-surface materials (both waste and ore), inherently leading to the disturbance of large surface areas of land over the life cycle of mining operations. With a greater focus on sustainable development, land currently and/or in the past subject to surface mining activities is increasingly being considered for future redevelopment opportunities post mine closure.

To enable such future development, any mined-out void needs to be reinstated to some workable final surface level through backfilling. The long-term settlement characteristics of such fills, which are typically of substantial thickness, becomes an important consideration for the future proposed development, especially as surface infrastructure can be severely affected by settlement of backfilled land. Therefore, controlled backfilling of mined-out voids is often a complex process and may result in settlement that continues for years after fill placement.

The purpose of this paper is to present a Risk Management Framework (RMF) to address excessive post-construction ground settlement in backfilled open-cut quarries, which should ultimately be incorporated in the mine closure plan. The benefits of developing and implementing such a RMF are the identification and management of any excessive post-construction settlement, as early as possible, within the area to develop. However, external factors such as dynamic economic market conditions, change in operational or land ownership and government

incentives to redevelop brownfield sites can all affect the initially identified post-closure land use. The approach within the RMF will differ depending on the timing of implementation, but the ultimate aim will always be to successfully reintegrate the mining area into the developed community after mining operations cease.

2 BACKFILLING

2.1 Closure Process

The closure process for open-cut mines involves a series of different stages, outlined in the Strategic Planning for Mine Closure (ANZAC/MCA, 2000):

1. Stakeholder involvement;
2. Planning;
3. Financial provision;
4. Implementation;
5. Standards; and
6. Relinquishment.

The closure plan may even identify different potential future land-uses (e.g. groundwater collection and treatment system, agricultural use, conservation areas etc.). However, it is the authors' experience that, unless a clear post-closure development plan has been agreed on, backfill material is often indiscriminately sourced and placed with insufficient engineering controls. This may lead to significant heterogeneity of the backfilled material due to variation in material properties (e.g. different mineralogical or chemical compositions, particle sizes, grading, etc.), as well as the degree of

engineering or modification achieved during placement (e.g. compaction, saturation, etc.).

2.2 Components of Backfill Settlement

Day and Wardle (1996) and Hills (1994) recognised that the total settlement occurring during and after placement of backfill material could be subdivided into four components, as indicated in Figure 1 below:

1. Immediate settlement;
2. Consolidation settlement;
3. Creep settlement; and
4. Collapse settlement.

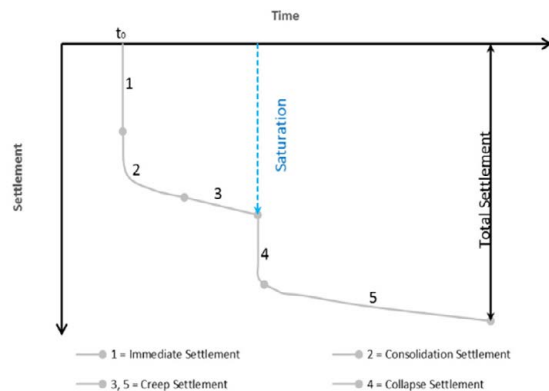


Figure 1. Components of backfill settlement (adapted from Day, 2013)

2.2.1 Immediate and Consolidation Settlement

Immediate settlement can be defined as “settlement under constant volume due to shear strains in the material as a result of the application of load”. This takes place instantly upon loading of the material (Day, 2013) and the strains experienced due to self-weight as the fill is being placed.

Consolidation settlement, also referred to as primary consolidation, can be seen as the (largely) plastic deformation of the backfill material due to the dissipation of excess pore pressures under self-weight. In the case of partially saturated granular material being used as a source of backfill, the development of excess pore pressure is limited (Day, 2013). Where cohesive material (i.e. material with a high fines content) and high moisture content are used during backfilling, consolidation will be time-dependent and a function of the slow (relative to free-draining granular materials) dissipation of water from the soil pores (Charles and Watts, 2001).

Immediate and consolidation settlements are generally not expected to have a significant impact on future developments above backfilled mining voids, as the backfilling operations typically takes many months to years to complete. Therefore, immediate and the majority of consolidation settlements are effectively “built out” (Day, 2013) during earthfilling operations and prior to redevelopment.

2.2.2 Creep Settlement

Hills (1994) states that the mechanism for creep settlement, also known as secondary consolidation, is one of gradual rearrangement of the material particles, resulting in a reduction in void ratio and increase in the density of the material. This behaviour occurs over long-term time periods under constant stress and environmental conditions, the rate of which diminishes with time (Charles, 2008).

Sowers et al. (1965), based on the analysis of several rockfill dams in the US, established a relationship between the amount of creep settlement under self-weight and the time elapsed since fill placement which still remains the most widely used, and accepted, method for predicting creep settlement of open-cast backfill (Hills, 1994). Long-term settlement monitoring has been carried out at numerous restored open-cast mining sites in the past (Charles, 2008; Day, 2013), to investigate the validity of the creep model, the engineering behaviour of the backfills and, ultimately, the suitability for building development. In all of these, the backfill properties are reported as heterogeneous and, due to the variability in environmental conditions at all study sites, the data gathered cannot be generalised and should only be regarded as indicative.

2.2.3 Collapse Settlement

Collapse settlement in open-cast backfill can be seen as additional settlement under constant total stress resulting from an increase in the water content of the backfill. Unlike other settlement mechanisms (e.g. consolidation or creep), collapse potential in fill does not automatically reduce with time. This is highly dependent on the fill material properties, particularly the density and moisture content at time of placement. Charles and Watts (2001) state that for fill placed dry of Optimum Moisture Content (OMC), collapse potential is predominantly a function of the ratio of dry density to maximum dry density (i.e. degree of relative compaction), whereas in materials placed wet of OMC the percentage air voids predominates collapse potential. Perhaps the most critical point is the fact that collapse settlement is possible in fill material compacted either wet or dry of OMC.

Field observation and research has shown that inundation of the fill with water is a major trigger of fill collapse settlement in opencast backfills. An increase in moisture content of the fill may occur as either the result of downward water infiltration from surface, through the old mine highwall, utility trenches, stone column ground improvement works and leaky drains / soakaways, or by a rising groundwater table following cessation of mine void dewatering operations (Day 2013; Hills 1994). This often represents a serious hazard for buildings on fill.

The magnitude of the settlement potential is typically expressed as a percentage of the thickness of fill material subjected to saturation. However, heterogeneity in the backfill material properties makes collapse settlement predictions difficult. Additionally, and unlike other settlement mechanisms, collapse

potential does not automatically reduce with time. It is therefore proposed that, unless it can be conclusively established that the fill does not have significant collapse potential, groundwater levels have returned to natural (pre-mining) levels or considerable collapse has already occurred, a potential risk of collapse settlement in the fill during or subsequent to building on the site should be assumed.

2.2.4 Differential Settlement

Rather than a separate component of the backfill settlement process, differential settlement manifests as a result of lateral variability in the magnitude and/or rate at which any of the aforementioned settlement components occur. This can take place due to a variety of factors, including variability in backfilling process (including differential stockpile surcharging), backfill material properties, underlying pit profile, pit boundaries and rate at which groundwater levels re-establish across the site. The resulting changes in surface settlement profile may lead to excessive angular distortion of surface structures.

3 SETTLEMENT RISK MANAGEMENT FRAMEWORK

By definition, a RMF is a structured process used to identify potential risks and define strategies for eliminating or managing the impacts to within tolerable limits. Its application to the challenges of integrating disused mining operations in land development areas can be complex and requires a careful balance of technical, commercial and planning aspects.

The proposed RMF discussed here focuses exclusively on the risk of excessive post-construction settlement for future infrastructure development on backfilled open-cast mining operations (Figure 2).

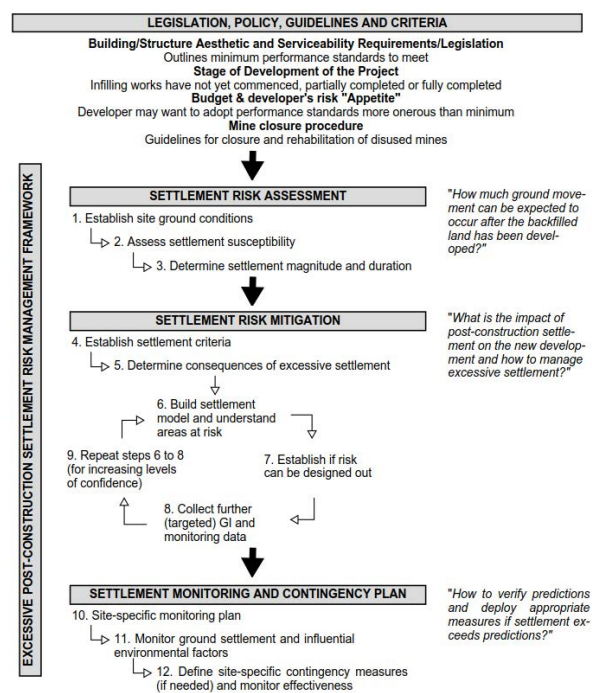


Figure 2. RMF stages of implementation

Note that, and as described earlier in Section 2.2.1, post-construction settlement is mostly influenced by the risk of creep and collapse settlement. As such, the RMF can be broadly divided into three separate stages of implementation:

- 1) Settlement risk assessment;
- 2) Settlement risk mitigation; and
- 3) Settlement monitoring and contingency plan.

Once future land-uses have been confirmed, a timeline to implement the separate stages of the RMF must be developed. Each project will have its own unique timeline, depending on the point in the operational life-cycle where planning for post-closure land development takes place, as well as planning legislation / administrative procedures.

3.1 Settlement Risk Assessment

The objective of the settlement risk assessment stage in the RMF is to answer the following question:

How much ground movement can be expected to occur after the backfilled land has been developed?

This requires an evaluation of the potential for excessive post-construction settlement to take place, its magnitude and duration. A thorough assessment of this risk requires an understanding of the potential severity and likelihood of each of the contributing settlement components (refer Section 2.2).

To arrive at a meaningful answer, several phases of geotechnical desk study and field investigations may be required, as the prediction tools are revisited with newer information.

3.1.1 Sampling and testing

When scheduling ground investigation works (GI) and associated laboratory testing, distinct approaches should be used, depending on the stage of development of the site:

- 1) Infilling works have not yet commenced;
- 2) Infilling works have been partially completed; or
- 3) Infilling of open pit has been completed.

The three scenarios listed above offer different degrees of certainty with regards to the characterisation of the backfill and assessment of its engineering characteristics, which require distinct sampling and testing regimes to inform the design.

In scenarios 2) and 3), ahead of scheduling GI, it is important to gather all available project data to date, which may include truck dockets, site notes, earthworks testing results and occasionally historical GI records, if applicable. However, this information is usually sparse and will likely need to be complemented with new data, which should specifically target settlement predictions. Depending on timeframes, a phased approach may be adopted, in order to balance the geotechnical model's accuracy with cost and time implications.

When scoping GI, an understanding of the key components of the proposed development is crucial, i.e. which assets are due to be completed first and/or are more susceptible to excessive settlement, to enable a targeted GI and not a widespread approach.

Furthermore, GI sampling techniques should be selected based on the likely testing regime. The latter should be preliminarily defined ahead of procuring GI to ensure the scheduled testing is not compromised by the available quality/type of samples.

3.1.2 Settlement susceptibility

A preliminary settlement susceptibility assessment can occasionally be made based on a desktop study. However, the available historical data and/or literature may not be (fully) applicable to the backfill materials in analysis and may require validation. Nonetheless, this high-level review can be useful when scoping GI, following which a first site-specific settlement susceptibility assessment can be completed.

This exercise should highlight the different types of settlement that the fill materials are susceptible to, their likelihood of occurrence and, just as importantly, a spatial indication of where these are likely to manifest. However, limited local data/published case histories may be available for some of the identified components of settlement (e.g. collapse settlement). In these instances, reliance on a statistical analysis of the GI results often provides a more accurate insight on the probability of occurrence, as opposed to literature-based correlations.

As a result, it is recommended that GI sampling and associated laboratory testing schedule strive to achieve a representative understanding of the fill properties at depth and across the site.

3.1.3 Settlement magnitude and duration

For the backfill materials identified as susceptible to settlement, the subsequent step is to predict its magnitude (usually providing a range of credible upper and lower bound estimations) and to forecast its occurrence over time. Note that some settlement components may not be time-dependent, but rather related to external factors. If any settlement components are estimated via empirical relationships from published literature (in the absence of GI results), careful consideration should be given to the uncertainty of extrapolating case history data to local fill characteristics.

Having estimated the magnitude and occurrence of the predicted settlement, a three-dimensional model of the site can then be established to help assess the projected total and differential settlement at the surface. The 3D model will typically comprise of a series of discrete soil blocks, each with a given set of settlement characteristics. Naturally, the sum of the accumulated settlements within each soil column provides an estimate of the total settlement at surface, at that given location. This discretisation is especially important, because for the majority of developments the key settlement hazard is not the total amount of

surface settlement, but rather the post-construction differential settlement experienced across site.

Therefore, for a successful development, it is crucial to develop a model that can not only compute the magnitude of settlement across the site, but also model its time dependency, so as to estimate a spatial distribution of the post-construction surface settlement at any given point in time. Naturally, the grid discretisation will be a function of the level of detail needed, as well as the amount of data gathered.

3.1.4 Large-scale field tests

Given the nature of GI, the information gathered from a finite number of data locations will be extrapolated to the wider fill materials. As such, and even if an expanded program of laboratory tests is completed (leading to a more complete and statistically significant set of results), there will always be a limit to the level of confidence of extrapolating data from relatively small selected samples to the entire fill. The latter will always present some inherent lateral and vertical variability in properties, when compared to naturally deposited soils.

A more representative behaviour of the infill materials can be obtained by undertaking large scale field trials, which can substantiate or disprove the predictions of actual post-construction settlement performance. This increased level of confidence in the settlement predictions does however require relatively larger levels of investment, as illustrated in Section 3.2.2.

3.2 Settlement Risk Mitigation

Once the risk of settlement has been assessed, measures can be developed and implemented to manage the risk of ground movement to the future infrastructure development. The settlement criteria, and subsequent preferred mitigation measures, tends to be a function of the proposed type of construction and its settlement tolerance. The latter is usually defined by the structure's allowable differential settlement and acceptable damage levels. Often, a maximum differential settlement of 1:500 across a building footprint is used, which is governed by aesthetic and serviceability requirements (Skempton and MacDonald, 1956). Other factors, including the time at which the RMF is developed, budget, risk appetite of developers and legislation, may all influence the preferred mitigation measures.

The impact of settlement on the new development can be graded using the following categories:

- **Inconsequential**, which will not impact on the performance of the (infra)structure/services and therefore does not impact on the structure design;
- **Significant**, which may either require some form of settlement mitigation measures to be implemented or a more detailed study of the proposed foundations behaviour ahead of structure design; or
- **Very significant**, where fill settlement mitigation measures will need to be specified and monitored ahead of structure design.

Understanding the severity of the hazard is therefore fundamental to help the development stakeholders assess the viability of a project as early as possible, as well as to enable expert geotechnical advice to inform on the civil and structural design of the works. This is especially relevant as the geotechnical component of the design (or as a minimum some preliminary advice) precedes other disciplines.

The time available for assessing the geotechnical properties of the site and/or developing mitigation measures needs to take that into consideration. Furthermore, any strategies put into place will likely require hold/monitoring periods to assess their effectiveness and corroborate the design predictions.

On the other hand, having a fixed date of completion results in a limited period of time over which mitigation measures can be monitored. As the latter may be further constrained by planning and financial restrictions, a staged approach to assessing the risk (i.e. likelihood and consequences) of settlement linked with incremental levels of investment may be advantageous.

This RMF proposes to address the delicate balance between confidence in the settlement prediction model and the level of investment by relying on the following decision inputs:

- 1) Construction programme with indicative durations of the fill settlement mitigation measures and associated monitoring;
- 2) Confidence in the settlement prediction model versus cumulative level of investment curves; and
- 3) Lower and upper bound forecast of the post-construction settlements, for a given level of confidence.

3.2.1 Construction programme and developer's business case

The development of the construction programme is essential as it informs the developer and designers on the time available to undertake geotechnical works. Given the importance of this discipline in these projects, an early geotechnical input in the decision process is crucial. Depending on the size and planning of the scheme, different approaches can be employed for distinct areas of the site.

Areas which may need to be developed first, often related to the financial viability of the entire development, may not allow for a phased and progressive approach to the development of the geotechnical model. In such case, the risk of excessive post-construction settlement can be designed out by employing flexible foundation systems and structure types which are not as susceptible to differential settlement. A similar methodology can be employed for high importance level structures within the development.

When a phased approach fits within the timescale of the development, it often yields a better return on investment for the developer, as it progressively enables new information to be collected to update and

enhance settlement predictions. This procedure allows for the improved accuracy of the settlement predictions to be weighed against the cumulative level of investment, which helps the developer gauge the financial impact of each stage of the works.

3.2.2 Confidence in settlement prediction model outcome versus level of investment

When a phased approach is employed, both the additional financial investment and the level of confidence in the settlement predictions can be revised at each stage of the assessment. Indicative model outcome certainty and cumulative investment curves are shown in Figure 3 to illustrate this staged progression. Naturally, each project will have its own set of curves and, depending on its business case and construction programme, the number of relevant steps/iterations may also vary.

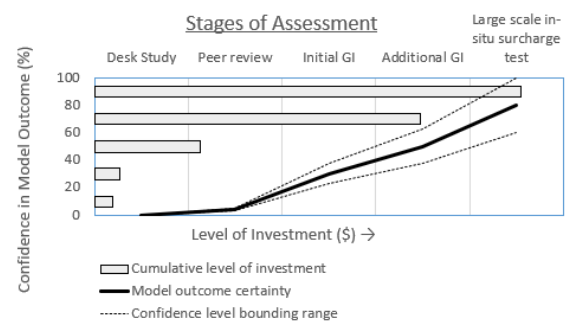


Figure 3. Example of confidence in model vs level of investment curves

Figure 3 depicts the effects of these incremental refinements (from the initial stage to proof of concept) in the overall level of confidence in the predictions.

Attention is also drawn to the peer review, at an early stage of the assessment, in order to corroborate the designer's views and provide confidence to the developer, whilst optimising the subsequent stages of design/assessment.

3.2.3 Settlement model outputs

To help inform decision makers on the likely risk of excessive post-construction settlement, a 2D visual output is proposed as the third decision input of the RMF (Figure 4).



Figure 4. Example of settlement risk areas for upper and lower bound estimates

Using this visual approach allows all relevant parties to understand the location of the development's risk areas, independently of their technical background. In turn, this information can help inform the most favourable building sequence, e.g. which components of the project should be prioritised or delayed, in order to allow for time dependent settlement to take place. Furthermore, this tool can also help delimitate the areas where remedial measures are needed and assist in targeting subsequent GI stages

For the best outcome, the settlement predictions must be revised as more information is gathered, narrowing down the range of settlement estimates and offering a higher degree of confidence in the modelling.

3.3 Settlement Monitoring and Contingency Plan

The final stage of the RMF is focussed on a) verifying predictions and b) defining appropriate contingency measures in case predictions are exceeded.

3.3.1 Monitoring

Settlement monitoring forms a critical component in evaluating and managing the residual risk to the future development and, due to the inherent uncertainty involved, predicting ground surface settlement "hot spots" in thick highly variable fill materials. Ultimately, it aims to provide the developer with a) timely notification of where and when predicted ground movements are likely to be exceeded and b) insight to the potential reasons for the observed exceedances.

A detailed, site-specific monitoring regime ideally should include a variety of ground deformation monitoring methods and technologies, which are currently available to the geotechnical engineer. Regardless of the specific measurement methods selected, the monitoring regime should ultimately be developed on a best-for-project basis which requires (amongst others) consideration of the proposed development and tolerance of structures to ground movement, site environment, access, budget, public perception and performance reliability.

It is important for any settlement monitoring program to not only focus on quantifying ground settlement, but also capture environmental factors that may have a direct influence on observed ground movements (e.g. groundwater levels and climate variations), as well as confirming that mitigation measures work as intended (e.g. subsoil drainage systems are free-flowing).

3.3.2 Contingency

Contingency measures form the final line of defence in the RMF, providing the developer with options to protect their assets from settlement-induced damage. The successful implementation of such measures relies heavily on the site monitoring program to ensure any actions are fit-for-purpose and implemented in a timely manner. As is the case with the settlement monitoring program, contingency measures need to be developed on a site-specific basis and agreed with all stakeholders prior to inclusion in the RMF.

4 CONCLUSION

The process to successfully rehabilitate disused open-cast mining operations as a feasible land option for future infrastructure development is a complex undertaking, and requires the input from a number of different specialist stakeholders to manage all associated risks. The RMF presented in this paper addresses a geotechnical risk often associated with such projects; the occurrence of excessive settlement of the backfill material post-construction. It is envisaged that this RMF will form part of a more comprehensive geotechnical, and ultimately overall risk management and mine closure plan.

The RMF provides a structured process to investigate and quantify potential risks due to backfill settlement, as well as identify potential strategies for eliminating or managing the impacts to within the limits tolerable to a particular project or developer. Each project will have its own unique timeline, depending on the point in the operational life-cycle where planning for post-closure land development takes place, as well as planning legislation / administrative procedures.

The approach of the RMF will differ depending on the type of post-closure development, where in the project life-cycle the RMF is implemented, the developer's risk appetite, as well as the market and environmental conditions. Nonetheless, the ultimate aim will always be to successfully reintegrate the mining area into the developed area after mining operations cease.

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Backfilled Quarry Development with Inbuilt Landfill Gas Solution

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ABSTRACT

A basalt quarry north east of Melbourne operated for over 30 years from the mid-1960s. Rock extraction occurred to a depth of approximately 28 m below ground level with dewatering. Since operations ceased in 1999, the quarry has been progressively backfilled with variably compacted 'clean fill' until 2017 and the groundwater level allowed to rebound close to its former level. Potential development of the land, for residential or mixed land use, triggered geotechnical and environmental investigations with the objective of identifying and managing below ground issues to support a planning rezoning process. Based on the investigation results, some of the key considerations for future development of the site were settlement of the backfilled quarry, foundation system options across the filled quarry extent and management of landfill gas. Development time, future ongoing management pressures and the need for large scale ground improvement, led to the consideration of a landfill gas control barrier integrated within the engineered fill ground solution, and an on-site trial. This paper provides the findings from the surcharge trial to date to combine the reduction in time to achieve an acceptable settlement of the fill and the management of entrapped landfill gas.

Keywords: landfill, settlement, gas, redevelopment, ground improvement, trial

1 INTRODUCTION

A 45-hectare parcel of land north east of Melbourne operated as a basalt quarry for over 30 years from the mid-1960s. The quarry footprint covers approximately 25 hectares, of which nearly 20 hectares is within the site boundary. Rock extraction had continued to a depth of approximately 28 m below ground level and was made possible through dewatering the local area. Between 1999 and 2017 the quarry has been progressively backfilled with variably compacted, but largely uncontrolled 'clean fill' and the groundwater level allowed to rebound unevenly but close to its former pre-quarry operations level.

Potential development of the land (currently zoned 'Industrial 1 Zone') for residential or mixed land use, triggered geotechnical and environmental investigations, with the objective of identifying and managing below ground issues and supporting a rezoning process, which includes completing an environmental audit.

Based on the investigation results, the key considerations for future development of the site were settlement of the uncontrolled backfill, foundation system options across the filled quarry extent and management of landfill gas. While individual solutions to these issues could be applied, including the use of surcharging to shorten settlement time of the uncontrolled fill, backfilling the remaining extent of the quarry footprint with engineered fill to design surface and installing a vapour barrier, several factors drove the design of a combined engineered solution and implementation of a trial.

This paper focuses specifically on the findings and solutions within the quarry footprint area of the site.

2 BACKGROUND

Due to the long history of quarrying and subsequent backfilling at the site, review of historical documents, including aerial photographs, and previous environmental and geotechnical investigation reports provided important data collected prior to and during the backfilling process. This information was supplemented by additional combined geotechnical and environmental investigations, over a period of two years, to infill and further minimise data gaps. Investigation included cone penetration testing (CPT), drilling of boreholes and their conversion to wells, installation and sampling of groundwater and landfill gas bore, test pitting, contaminant and geotechnical analysis.

The site is underlain by Quaternary Newer Volcanics basalt, of which at least four lava flows were quarried down to within 2 m of the base of the unit. Groundwater flows through this unit. The basalt, approximately 30 m thick, is underlain by an equivalent of the Tertiary Werribee Formation. This 5 m to 6 m thick layer of sandy clays, sands and gravels contains a confined aquifer and overlies Silurian siltstone. An influent creek runs along the western and southern boundaries of the site.

2.1 Environmental Issues

While the site was considered to be backfilled with 'clean fill' and there was no evidence of putrescible waste, past Environmental Protection Authority Victoria (EPA) inspections noted uncontrolled dumping of building rubble, confirmed when concrete rubble was encountered during a recent site investigation. In 2001, EPA granted approval for the site to accept Coode Island Silt (CIS), which is either a potential or actual acid sulfate soil (ASS). The EPA stipulated that it was disposed below the rebounding water table within the quarry, at that time. Based on

this information the following issues required consideration in formation of a development solution:

- Fill materials – potentially contaminated imported soils, presence of building rubble and other anthropogenic materials - contaminants within the imported fill potentially could impact future site users and/or leach to groundwater.
- Fill materials – ASS producing sulfuric acid and hydrogen sulfide, if placed above water table / exposed to air.
- Groundwater – potentially impacted by contaminants within the fill
- Landfill gas – presence of organic matter within fill biodegrading to produce methane and carbon dioxide – requirement for landfill gas risk assessment and potential engineered solution for site development.

2.2 Geotechnical Issues

The prolonged history of quarrying and backfilling practices has led to multiple snapshots of information at specific times from different sources and for different outcomes. Additional site investigation was undertaken to refine the understanding of the following geotechnical issues encountered:

- Quarry footprint extent – the lateral extent of the quarry changed over time, with voids being filled while other areas were still being quarried. The understanding of its extent is extremely important for the design and construction of any future development.
- Benching – the sides of the quarry were benched with access ramps / haul roads. Benches varied in width, length, relative height and near vertical slope of the quarry wall.
- Quarry floor – the base of the quarried surface was variable, with two deeper 'pondage' areas.
- Fill materials – the acceptance of varied soils (clay, silt, gravel, boulders) with different properties leads to differential settlement and varied response to moisture, plus building rubble and biodegradation of timber. The localised presence of imported Coode Island Silt adds a compressible soil type. It was reported that CIS was deposited close to the liquid limit, with rubble added to the soft sediment for trafficking.
- Fill placement – it is understood that while some compaction of fill layers occurred, formal compaction and moisture conditioning of material was not general practice. Filling by tipping from the benches of the quarry and edges of the quarry occurred during quarrying, and from the edge of the quarry later, during backfilling period.
- Groundwater – water level affected by rebound from dewatering and potential creek water ingress. Highly variable levels measured within fill with shallow water levels limiting ground improvement options.

3 DEVELOPMENT

Over the investigation period, the development drivers changed. Time, cost, full scale versus staged development, environmental assessment and audit, requirements for rezoning, proposed mixed land use, future ongoing management pressures and the need for large-scale ground improvement, all played roles in the direction of development planning progress. The potential to manage both environmental and geotechnical site issues with combined engineering solutions, rather than dealing with each issue separately, allowed the projected development timeline to be reduced.

For the purposes of the project objective, the end use is assumed to be mixed use with low density residential (access to surface soils) and open space.

4 KEY SOLUTIONS

The key issues were considered to be high total and differential settlement and potential ongoing landfill gas management. Various solutions were assessed over time, a selection of which are described below.

4.1 Ground Improvement

The ability to construct buildings and infrastructure on the filled quarry area relies on preventing or limiting settlement of the structures. A piled option had previously been investigated but was deemed costly, with potential problems associated with deflection of piles on quarry walls, impenetrable waste and punching through the base of basalt. An alternate option to improve the properties of the existing ground was considered. Ground improvement could enable construction on shallow foundations by the reduction of long term settlement. While there are a number of different methods the following was considered for adoption.

4.1.1 Surcharging

Ground improvement through surcharging was considered an option to reduce settlements to acceptable levels but differential settlement potentially remains an issue, particularly across the boundary between filled profile and natural ground outside the quarry extent. This was proposed to be accommodated through planning and engineered solutions. Surcharging can take considerable time and requires large volumes of material to be imported to site. A moveable surcharge that covers only part of the area was preferred but relies on the settlement being reduced over a shortened period. Given the heterogeneity of the existing backfill materials and variability of past compaction, shown through the CPT data, the rate of settlement was estimated and modelled but was deemed to have a low level of accuracy and a trial was considered to be essential.

4.1.2 Wick drains

Wick drains, also known as prefabricated vertical drains, were considered and included in the trial as they have a number of advantages:

1. They allow any excess pore pressures to dissipate more quickly and accelerate settlements
2. They accelerate equilibration of the groundwater levels within the fill and accelerate hydro-compaction
3. They allow the rapid release of landfill gas pockets both during installation and post settlement

Wick drains are installed in a grid pattern. Drainage, storage and management of significant volumes of expelled (and potentially contaminated) porewater is required to be considered.

4.2 Engineered Fill Finished Level

The level of backfill at the time of the investigation, relative to proposed future development masterplan levels, required approximately 3.5 m of additional engineered fill to be placed. The reasons for this were:

- To raise the ground level to match the ground level outside of the quarry footprint, allowing drainage and civil design (with consideration for additional settlement).
- The existing fill is not currently a suitable geotechnical founding material for future development (uncompacted).
- A verified clean fill cap is likely to be required by the EPA appointed Auditor for low density residential use due to contaminant impacts in existing fill.
- Provision of a low permeability clay cap (see landfill gas – Section 4.3).

While it is possible that some materials present on site could be utilised as engineered fill, the majority of the required volume of material would need to be imported, meeting both geotechnical and environmental specifications.

Optimally, a clay-rich fill would form a low permeability, 2 m thick layer over which a non-reactive fill material, such as mudstone is placed in the upper 1.5 m to reduce the ground surface shrink-swell seasonal reactivity.

4.3 Landfill Gas

Landfill gas investigation at the site to date has measured elevated methane and carbon dioxide concentrations within the quarry extent, with lower concentrations outside the quarry; inferring that the quarry fill is the source of methane generation. Measured flow rates were generally low and the Gas Screening Level calculated within the quarry area, determined a Characteristic Gas Situation 2 (low risk) in accordance with BS 8485:2015. A combination of engineered gas protection measures

are typically integrated into residential floor slabs with under-floor ventilation and a gas impenetrable membrane. An alternate passive venting system option at depth creates a pathway for gas to disperse beneath a low permeability clay layer, providing one engineering measure across the site, with limited risk of damage during construction or over the life of the development. While the option may not necessarily mitigate all of the requirements for engineering measures within individual dwellings, it can significantly reduce the measures.

5 PROPOSED COMBINED SOLUTION

The proposed solution integrated the required ground improvement and need to raise ground level with the management of landfill gas.

The following sequence of elements which integrate to form the combined solution are presented:

5.1 Subgrade preparation

Preparatory earthworks are required to form a compacted earth mound (Figure 1). The aim is to raise the existing backfilled ground surface at the centre of the loaded area, ensuring a fall of 4% from the centre to the perimeter of the area when placed. This allows for a conservative minimum 1% fall to be retained following long term settlement. The gradient is required to ensure flow through the subsequent permeable layer for the pore water and then the landfill gas, albeit in different directions.

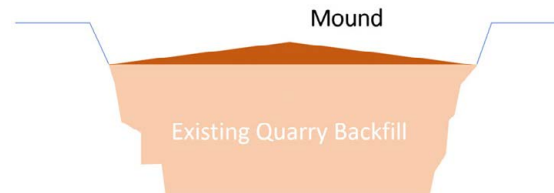


Figure 1. Indicative mound preparation (exaggerated)

5.2 Wick drain installation

Wick drains are installed through the mound material and the underlying existing backfill to the base of the quarry (Figure 2) providing a pathway for water and gas to travel to the surface in addition to equilibrating the groundwater level within the quarry backfill. A grid was proposed across the development area, to be determined based on required settlement timeframes. The investigation encountered construction debris in the fill, which was anticipated to cause early refusal for wick drain installation. Pre-drilling was incorporated into the conceptual methodology, but was found during construction not to be required. The top of each wick drain integrated into the permeable layer. Monitoring equipment such as extensometers, vibrating wire piezometers and settlement plates are installed at this stage to allow monitoring of settlement and pore pressures over time.

5.3 Permeable Layer

The permeable layer, installed over the mound, is required to ensure flow of expelled porewater (via wick drains) to the perimeter and also ensures landfill gas can migrate up gradient to venting infrastructure.

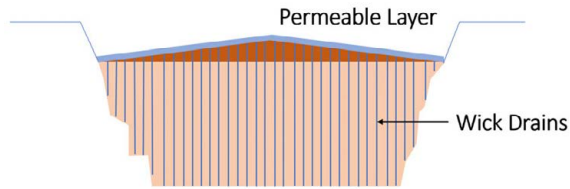


Figure 2. Indicative wick drain and dual purpose permeable layer (exaggerated)

The permeable layer is made up from two elements as described below.

5.3.1 Gravel

A 350 mm layer of gravel is placed across the mound surface with each wick drain exiting into the gravel layer. Flooding of this gravel layer must be prevented.

5.3.2 Piping and Geotextile Layer

Vertical gas piping infrastructure (Figure 3) is installed through the gravel layer at this time, positioned towards the highest point of the mound to allow venting of the rising landfill gas with sufficient grade to avoid landfill gas entrapment. Lateral drainage pipes at the perimeter of the permeable layer are installed to divert water to sump locations. A non-woven geotextile layer is installed over the gravel to prevent fines from subsequent engineered fill material migrating into the gravel layer, which could reduce the hydraulic conductivity of the drainage layer.

5.4 Engineered Fill

The engineered fill layer is made up of two sub-layers, both to be placed and compacted as per guidelines for earthworks (AS 3798-1996). The lower layer forms a low permeability layer at least 2 m thick to inhibit migration of landfill gas and is to be constructed using suitable clays. The upper layer provides a geotechnically suitable low-reactive subgrade on which residential development can be founded (Figure 3).

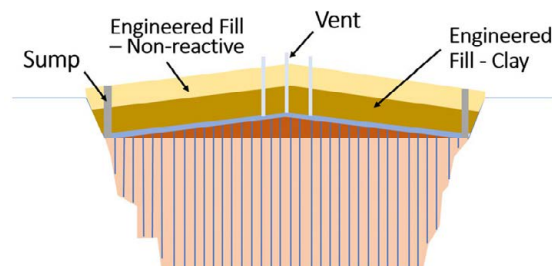


Figure 3. Indicative engineered fill layers (exaggerated)

At this stage sumps will need to be constructed around the perimeter to collect porewater and any surface water ingress, which can be pumped to surface, and managed. These sumps may be utilised within the future development.

5.5 Surcharge

To accelerate settlement of the existing fill, placement of surcharge material is required over the engineered fill layer. The height and volume of surcharge required for the development is governed by the desired rates of settlement and associated development timeframes, allowing for primary consolidation, secondary compression (creep) and, hydro-compaction. A 6 m high surcharge load (Figure 4) was originally anticipated to be in place for 1.5 to 2.5 years depending on the depth of the quarry and adopted settlement tolerances.

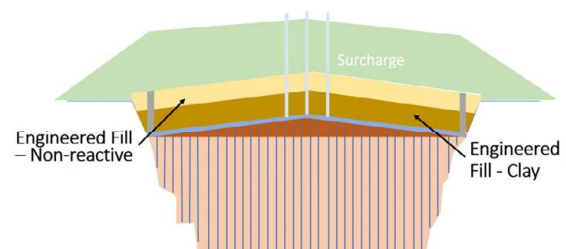


Figure 4. Indicative surcharge (exaggerated)

The gradients and volumes of fill layers and surcharge required to implement the solution across the entire quarry extent at one time was likely economically and practically unfeasible. Typically, development on this scale occurs in stages. By dividing the site into stages, a 'rolling' or movable surcharge could be re-used across the staged areas over time. Development of initial porewater drainage and ongoing gas venting infrastructure would be required per stage, taking into account the timeframes for settlement per stage and effects on the adjacent / subsequent stage. Additionally, design and preparation of the permeable layer 4% fall will need to consider each development stage and the quarry development as a whole.

The conceptual solution was presented to the site EPA appointed environmental auditor to ensure that the proposed solution was reasonable, with respect to obtaining regulatory approval in the future. The preliminary review confirmed that the sequence of works appeared reasonable, subject to further investigation and monitoring, including conducting a trial.

6 SOLUTION TRIALLED

The focus of the trial was to demonstrate that the magnitude and timeframe for settlement modelled was realistic for the purposes of planning future development, while also testing the conceptual integrated solution.

The conceptual solution was applied on a reduced scale. A 50 m by 50 m trial area was selected over the deepest area of the quarry, partially across a former 'pondage' area backfilled with compressible Coode Island Silt early in the backfilling history of the quarry.

6.1 Subgrade Preparation

Available on-site material most recently used to form a working platform in an alternate portion of the site was used for subgrade preparation. The 4% gradient mound was constructed to the furthest extent of the proposed surcharge batter footprint and density testing confirmed the specified compaction parameters. At this time, swale drains were also excavated around the perimeter of the surcharge extent to capture runoff from the trial area and also to prevent surface run off from the rest of the site affecting the trial. This was designed to drain to a peripheral stormwater pond outside of the area so as not to potentially impact trial measurements. Unfortunately, this failed at various times.

6.2 Wick Drains

Wick drains were installed on a grid pattern at 1.5 m spacing. The filled quarry level at the trial pad stands approximately 3.5 m below surrounding ground level therefore fill was anticipated to be approximately 24.5 m thick. During the installation it was ascertained that pre-drilling was not required. Of the 1140 wicks installed, 99 were replacements for wick drains that refused on shallow solid inclusions within fill, or the baseplate (which fixes the base of the wick drain at the installed depth) refused to hold (potentially caused by a void). 60% of wick drains were installed to 24.5 m depth or greater. Gas and water were noted to be expelled from some of the wick drains on completion of installation. This was most likely linked to pockets of gas being released.

6.3 Permeable Layer

The gravel layer was installed with the associated drainage pipes. These were linked to two separate areas of connected leachate collection tanks (IBCs) with the purpose of measuring the volumes of porewater expelled during settlement. In the trial the geotextile layer was installed wrapped under the outer edge of the gravel layer and perimeter drainage pipe, to ensure the drainage pipe retained its position once the surcharge had been applied. During each monitoring event the volume of porewater was measured, with tank content pumped to a distant managed storage pond when required.

6.4 Surcharge

Monitoring of settlement, from the top of the mound layer below the gravel layer was the focus of the trial. As the engineered fill layer was not required for the surcharge trial, this layer was replaced with 3 m additional surcharge material. In total a 9 m high layer of surcharge material, totalling approximately 40,000 m³, was added to the trial area, with 1V:2H to 1V:2.5H batters and a ramp for plant access. The

import and placement of fill was conducted over one month and was built up in stages, filling the southern side of the trial area by a few metres, followed by the northern area, for each stage. This occurred over a longer period than originally anticipated and modelled. However, the time period adopted can be considered potentially more realistic of future site development.

The full surcharge load remained in place for three months upon which the primary consolidation was mostly completed, after which the top 6 m of surcharge was removed, replicating the removal to the top of engineered fill level of the conceptual solution. Unloading was conducted over a period of four weeks.

6.5 Monitoring

The monitoring equipment was installed within the 50 m by 50 m trial area and consisted of:

- Two vibrating wire piezometers each recording porewater pressure at two depths.
- Two extensometers, each with a plate magnet and three spider magnets positioned at various depth intervals.
- Five settlement plates.

Baseline monitoring occurred on installation of the permeable layer. The frequency of monitoring was initially weekly, dropping to fortnightly during the final month of the full surcharge load period, with additional monitoring during the rebound period, during and following unloading. Monitoring reduced to monthly and then every two months after unloading and as settlement reduced.

7 FINDINGS

7.1 Settlement

The total settlement measured over the period to date is between 522 mm and 625 mm, including the period in which the site was surcharged (refer Figure 5).

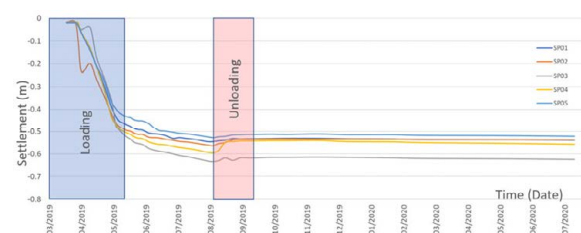


Figure 5. Trial settlement plate results

This is in the order of 2.8% of fill depth. As expected, this is greater than the 1% to 1.5% measured in similar quarry environments in engineered fill without surcharge (Colls et al 2010). Given the variable groundwater level at the site, the upper 5 m of fill has the potential to settle through hydro-consolidation prior to and during surcharging, as the water level equilibrates. Hydro-consolidation is documented to contribute settlement of between 3% and 6% of fill

depth of non-engineered stiff clay fills and fill with low levels of compaction (Waddell and Wong 2005) (Charles and Watts 2002)

Upon unloading, a rebound of up to 150 mm occurred followed by settlement over the subsequent period of six months of typically about or less than 5 mm (Figure 5) prior to flooding.

Heavy rainfall combined with a pump failure resulted in flooding, saturating the fill, entering the wick drains and imposing additional load not anticipated nor modelled. This has impacted and slightly increased the recently measured settlements and possibly initiated hydro-compaction of the fill above the water table.

Extensometers confirmed the settlement at the targeted depths was apportioned relative to those depths, with no significant variations in responses which could be attributed to different fill types, such as Coode Island Silt.

7.2 Porewater

Porewater was expelled via wick drains and drained through the permeable layer to the perimeter drain and collection tanks, as anticipated. The volume of porewater expelled is proportional to settlement. Initial high levels of settlement resulted in high volumes of porewater being collected. During the loading event issues with measuring the volume of porewater were encountered.

Vibrating wire piezometers, measuring porewater pressure, were severed during earthworks. Rectification works were successful in repairing three of the four inputs. The remaining readings from both upper and lower sensors showed that the pore water pressures stabilised once the surcharge was removed albeit at a higher level than prior to surcharge.

7.3 Gas

No venting infrastructure was installed to monitor gas flow or concentrations during the trial. However, landfill gas was noted to be expelled during wick drain installation, potentially penetrating pockets of gas trapped by layers of clay or other low permeability materials in the fill. Landfill gas will be monitored at a later stage.

8 CONCLUSION

The ground improvement trial confirmed that installation of wick drains and surcharging together are effective ground improvement techniques for use on this site to accelerate fill settlements and provide a pathway to displace landfill gas within the quarry fill. The period of primary consolidation (and hydro-consolidation) was shorter than predicted for the adopted wick drain spacing and will mostly occur over a three month period. The ability of wick drains to accelerate rebound of the groundwater table within the quarry fill also speeds up the process of any collapse settlement.

Variability of settlement results across settlement plates and extensometers is indicative of the potential for considerable differential settlement. The trial results aid the ability to build in contingency for differential settlement in the calculations for percentage fall of the preparatory earthworks mound at its surface. This will ensure a flow gradient is retained post loading in the dual purpose permeable layer.

A 2 m low permeability clay-rich barrier over the permeable layer provides a gas protection measure, which will prevent or at least greatly inhibit gas migrating into future dwellings above. The dwellings might still require a reduced landfill gas engineered mitigation measure below the foundations, depending on the EPA-appointed Auditor decision. The top of this clay-rich engineered layer is at least 2 m below the typical civil and residential construction works at or above the original site ground level, together with associated services, reducing likelihood of a breach of the gas protection measure. Basements and swimming pools are likely to be precluded. Further landfill gas monitoring is required to confirm the Characteristic Gas Situation for the environmental audit.

9 ACKNOWLEDGEMENTS

We acknowledge the input and support of our client in the preparation of this paper. The project is confidential at this time.

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Brown gold: redevelopment of former quarry and landfill sites

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ABSTRACT

This paper presents a discussion of key geotechnical considerations and challenges associated with the rehabilitation and redevelopment of former quarry and landfill sites, including open quarry pits to be backfilled with imported engineered fill and former quarry pits that are already backfilled with uncontrolled fill materials where it is not practical to remove the landfill/uncontrolled fill materials as part of the rehabilitation/redevelopment. Geotechnical considerations and challenges that are discussed include the importance of communication with regulatory authorities throughout planning, design and handover, the challenges associated with geotechnical investigation of former landfill sites, the availability of materials from onsite sources or importation for use as engineered fill, requirements for assessing and monitoring ground settlement and the importance of quality assurance (e.g. Level 1 inspection and testing) during rehabilitation/redevelopment earthworks. Development outcomes ranging from public open space to residential or commercial development are also discussed.

Keywords: quarry, landfill, brownfields development

1 INTRODUCTION

The practical challenges and costs associated with the redevelopment of former brownfield quarry and landfill sites have historically limited their development potential. However, as Melbourne's population and land value increase, former quarry and landfill sites are increasingly being targeted by a range of public and private developers, with potential associated benefits for community amenity and the sustainable reuse of in situ materials (e.g. stockpiles of uncontrolled fill).

This paper presents a discussion of key geotechnical considerations and challenges associated with the rehabilitation and redevelopment of former quarry and landfill sites, based on the authors' experience across a range of Melbourne sites over the past 20 years. Each site typically presents a unique set of challenges but for ease of description the following simplified groupings have been adopted in this paper:

- Open quarry pits, or pits with a shallow depth of uncontrolled fill that can be removed to expose natural subgrade materials, where there is the opportunity to control the nature of the backfill materials to target a specific development outcome.
- Former quarry pits that are already backfilled (typically with uncontrolled fill materials ranging from soil and rock to municipal waste), where it is not practical to remove and replace the backfill and the nature of the existing conditions and the potential for ground improvement will govern the geotechnical approach for development.

Development outcomes ranging from public open space to residential or commercial development are also discussed.

Whilst land contamination issues are a key aspect of the potential development of former quarry and

landfill sites given there is typically a requirement to complete an Environmental Audit, these issues are not discussed in detail in this paper which focusses on geotechnical considerations in particular the settlement characteristics of filled sites and the resulting implications for development.

2 FEASIBILITY AND CONCEPT PLANNING

Developing a ground model that relates the expected ground conditions to the type of development proposal and a remediation strategy that clearly communicates how the land may be developed is critical to the conceptual planning of quarry/landfill developments and to inform feasibility or due diligence assessments. However, feasibility assessment will not be limited to geotechnical considerations and related factors such as the likely site value pre- and post-development (likely influenced by the site location), the level of community support for development, and any environmental or planning restrictions on the type of development (including Earth Resources Regulation – the Victorian Mining Regulator – Work Authority or rehabilitation requirements) will need to be considered by the relevant specialists.

Quarry or landfill developments will often require rezoning to be carried out before development can begin in earnest and a key early challenge for the project team can be in balancing the need to obtain sufficient baseline geotechnical information to support planning applications with project budgets that may be limited until there is confidence that planning approval will be granted. There is therefore typically an initial desktop or non-intrusive phase of assessment where available information about the site history and likely geotechnical requirements for development are subject to preliminary evaluation.

Key geotechnical characteristics to be considered in the initial assessment of quarry and landfill sites for development include but are not limited to:

- The depth and extent of the former quarry pit(s), e.g. a relatively shallow, wide pit or a relatively deep, narrow pit.
- The geological setting e.g. a hard rock/basalt quarry, a soft rock/siltstone/brickworks quarry, or a sand quarry.
- The depth to groundwater and whether ponded water is present within the pit.
- The nature of any backfill materials within the pit (e.g. whether they were placed as controlled or uncontrolled fill and the type of waste materials likely to be present) and whether it is practical to remove them as part of the proposed development.
- The history of development, e.g. the timing of quarry extraction and backfilling.
- The extent of natural ground or generally undisturbed areas outside the former quarry pit(s), which if present offer the potential for conventional development to offset the challenges associated with the quarry pit(s). The respective characteristics of different parts of the site should be considered in planning the type of development proposed.
- The surface topography, which can be particularly important for former landfill sites if they are mounded to facilitate surface water runoff and are subject to restrictions on levelling earthworks to avoid exposing waste.
- For former landfill sites, the presence of surface, side or base liners that may restrict development.
- The availability of earthworks materials, e.g. whether material stockpiles or potential borrow areas are available onsite or imported fill materials are required.
- The presence of buildings or underground structures such as tunnels associated with quarry or other operations that may affect development in areas outside the pit(s).

The following broad categories of development are considered in this paper:

- Public open space, i.e. parkland and sports fields, which generally have a high tolerance for ongoing settlement. However, there may be associated buildings (e.g. sports pavilions) and infrastructure such as light towers that have a low settlement tolerance and there may be some types of sports fields that have a relatively low settlement tolerance, for example synthetic surfaces and sports fields for elite use.
- Low-rise residential development, i.e. 1-2 storey residential buildings which could potentially be supported on shallow footings founding in suitably engineered fill materials.
- Medium to high rise development, i.e. buildings of more than about 3 storeys for which it is unlikely that fill materials are a suitable founding stratum, and where piled footings founding in natural materials below fill are likely to be required.
- Infrastructure such as roads and underground services associated with the above broad development categories.

The geotechnical challenges associated with former quarry and landfill sites mean the planning process is typically more complex than for other greenfield and brownfield sites. Early engagement with Council or other planning authorities to clearly communicate the geotechnical challenges and the proposed redevelopment strategy in the context of the site-specific geotechnical characteristics can be beneficial to provide Council with confidence that development is feasible. Consideration may need to be given to establishing an owners' corporation to manage public assets such as road reserves to assuage Council concerns in relation to asset management and maintenance. There may also be more onerous bonding and maintenance period requirements for assets that will eventually be transferred to Council ownership in consideration of the potential for ongoing settlement. It is also critical that development details (e.g. final surface levels and the general type of development proposed) are established early as there may be limited scope to make adjustments once bulk earthworks or other development works commence.

The following sections of this paper provide more detailed discussion of the key geotechnical considerations and challenges as they relate to open quarry pits (where the development can influence the backfill materials) and former quarry pits that are already backfilled.

3 OPEN QUARRY PITS

3.1 General

The key distinction made in this paper with respect to open quarry pits is that there is the opportunity for the geotechnical design of earthworks strategies (backfill) to accommodate the proposed development.

The design of backfill earthworks should consider the Australian Standard AS3798 (2007) 'Guidelines on earthworks for commercial and residential developments', noting that AS3798 (2007) provides guidance on minimum considerations and not explicit requirements. Reference to the Standard is not a substitute for a technical specification that sets out the *specific* requirements for a *specific* development at a *specific* site. It is critical that the geotechnical design of earthworks takes the site-specific geotechnical characteristics and nature of the proposed development into consideration.

In this paper, 'controlled fill' (also referred to as engineered or structural fill) is used to refer to fill placed with engineering supervision in accordance with a specification developed in accordance with the guidance in AS3798 (2007). For controlled fill it is important that the specification is in place prior to the commencement of earthworks so that compliance with the specified requirements can be assessed. Fill materials placed without engineering supervision or where there are no records of the engineering supervision having been undertaken are considered 'uncontrolled fill' materials.

If low-rise residential development is proposed and requires a site classification in accordance with AS2870 (2011) 'Residential slabs and footings', reclassification (from Class P) cannot take place unless the fill is controlled. However, there may be circumstances where the effort to place controlled fill is not justified by the proposed end use, for example the development of parkland may not require the use of controlled fill (potentially with the exception of near surface zones).

A key distinction between controlled and uncontrolled fill materials is that the specified requirements for controlled fill materials and associated records of compliance provide a technical basis for estimating the future engineering performance of the fill based on reasonably consistent and predictable material properties. Whilst the specific material requirements are critical for a specific development proposal, if the nature of the development changes the presence of controlled fill materials still provides a basis for assessing the likely ground response. If the fill is uncontrolled or if the specified requirements are overly broad, the potential for variability makes it difficult to assess the likely engineering performance of the fill.

3.2 Geotechnical design of backfill

Geotechnical issues to be considered in the design of backfill earthworks include:

- The required engineering performance of the fill platform, including where relevant to the development proposal:
 - The timing and magnitude of settlement of the fill platform.
 - The reactivity of the fill (i.e. shrink/swell movement due to seasonal moisture content changes).
 - The potential for differential settlement at the interface between filled areas and areas that have not been filled.
 - The ability of the backfill to support the proposed structures (i.e. the bearing capacity of the backfill).
 - The suitability of the fill as subgrade for pavements, and as the zone in which future trenching and excavation works (e.g. for underground services and building footings) will occur.
 - Aesthetic considerations for the completed backfill surface.
 - The nature and strength of the subgrade material at the base of the pit, including whether water (from groundwater or surface water inflow) is present and whether there are existing uncontrolled fill materials within the pit that need to be removed before backfilling can commence.
 - The geotechnical properties of the materials to be used as backfill (e.g. particle size distribution/grading, clay content and plasticity). These properties will affect the engineering performance of the backfill.
 - The availability and cost of site-won or imported backfill materials and whether processing of the materials is required.
- The target level of compaction and moisture content during the placement of backfill. In general terms, the higher the level of compaction achieved during backfilling, the smaller the amount of settlement once backfilling has been completed. The fine fraction of backfill should be placed and compacted at a moisture content that is close to its optimum moisture content for Standard compaction (SOMC). Fill that is poorly compacted or placed significantly dry of SOMC has an increased risk of collapse settlement as it becomes saturated (e.g. due to recovering groundwater levels).
 - The level of engineering supervision (including survey control and monitoring of the performance of the backfill) during and after the backfilling earthworks, i.e. Level 1 inspection and testing or Level 2 sampling and testing as set out in AS3798 (2007).
 - The potential impact of groundwater on construction processes and the long-term performance of the fill. There is the potential for groundwater levels to change as a result of backfilling.
 - The stability of quarry pit batter slopes.

For deep controlled filling in former quarry pits it is typical to adopt different zones or types of fill at different elevations within the pit to accommodate the potential variability in fill sources and to address issues such as groundwater control in the base of the pit and aesthetic requirements near the final surface. In general, at least three distinct vertical zones may be required within the backfill platform, including:

- A zone of open-graded backfill at the base of the quarry pit to help control water inflow.
- A zone of 'general' backfill placed over the open-graded fill zone (with a geotextile separation layer between these zones).
- A surface zone of nominally 'higher quality' fill (e.g. fill with stricter controls on grading and compaction) to help bridge over variability in the nominally 'lower quality' general fill as well as to provide suitable material for surface development works.

Colls et al (2010) provides an example of the zoning approach used during the backfilling of the former Niddrie quarry (refer Figure 1). Charles (2008) provides several useful case studies relevant to deep controlled fill sites and approaches for estimating settlement. However, given the potential for variability in the fill materials actually used as well as the methods for placing fill and the level of compaction achieved, settlement monitoring (e.g. via survey of settlement monitoring plates or other monitoring instruments installed at different depths within the fill platform) and interpretation of the monitoring data are usually required to assess the settlement performance with respect to initial estimates. Given the rapid rate at which site conditions can change during bulk earthworks, survey control of the extent and thickness of approved subgrade and controlled fill cells is also a key component in quality control to provide

confirmation that there is vertical and horizontal overlap between controlled fill cells.



Figure 1. The former Niddrie quarry prior to redevelopment (top, c.2001) and in November 2019 (bottom)

3.3 Geotechnical investigation

Intrusive geotechnical investigation for open quarry pits may be limited to test pits or boreholes to assess the nature of potential borrow materials and likely processing requirements, the thickness and extent of uncontrolled fill to be removed, and groundwater considerations. Other information such as historical aerial photographs (including photogrammetry) and quarry records if available can assist to provide site conditions information.

Where Level 1 inspection and testing by a geotechnical inspection and testing authority (GITA) is specified, a completion report should provide confirmation that the specified requirements have been met but may still not be sufficient to inform design development. Once the backfill earthworks are completed, there will usually be a requirement for additional geotechnical investigation (e.g. boreholes for in situ testing to assess the strength of the fill) to inform design works. If the specification was developed for the eventual development proposal and the earthworks complied with the specification, the post earthworks investigation should predominantly be a confirmatory exercise e.g. to confirm the site classification in conjunction with a review of relevant engineering records and the results of settlement monitoring. Where reclassification of controlled fill is required it will usually be necessary for the GITA to provide the reclassification.

3.4 Common issues for development of open quarry pits

Common geotechnical issues observed in bulk controlled earthworks projects that are relevant to the backfilling and development of open quarry pits in general include the following:

- Where a significant volume of imported fill is required the availability and cost of suitable

imported fill materials can present significant project risks, as they depend on market factors and the geographic location of the destination site with respect to potential source sites. Where imported fill materials are required it is important that the development strategy and specification are developed recognising the likely commercially available fill materials.

- Inadequate specification or the lack of a specification leading to unsatisfactory performance, e.g. excessive swelling of high plasticity clay compacted significantly dry of SOMC. The development of a specification that is targeted at the specific site conditions and development proposal is a key factor in successful bulk earthworks.
- Misinterpretation of the requirements of Level 1 inspection and testing, which AS3798 (2007) defines as the presence of the GITA at all times during earthworks activities. Periodic visits to undertake compliance tests represent Level 2 sampling and testing. However, there appear to be differences in the industry in how the requirements of Level 1 are interpreted.
- Responsibility for engineering quality control. There is the potential for a conflict of interest if the earthworks contractor is responsible for their own quality control (e.g. via engagement of the GITA). The authors see it as important that the Principal or owner of the development is responsible for engaging independent engineering quality control.
- Difficulties in achieving compaction against steep pit walls. Quarry pit walls especially in hard rock quarries can be near vertical and it can be difficult achieving consistent and adequate compaction in the zone adjacent to the wall leading to excessive total and differential settlement. It may be necessary to reduce the pit slope (particularly near the finished surface level) to reduce the potential for differential settlement across this interface.

4 BACKFILLED QUARRY PITS

4.1 General

The key distinction made in this paper with respect to backfilled quarry pits is that the backfill materials are already present and must be accommodated in the proposed development works (i.e. not removed and replaced with controlled fill). Where backfill materials are already present (and placed by others) they typically represent uncontrolled fill.

The potential for significant ongoing total and differential settlement of uncontrolled fill materials is usually the key geotechnical consideration when assessing redevelopment options for these sites. Differential settlement is generally a maximum at the interface between fill and natural ground materials and hence it is important to define the boundary of the backfilled former quarry pit. There are significant risks associated with attempting to support buildings or infrastructure with a low settlement tolerance directly on uncontrolled fill, as illustrated by the Yarraville sinking village case study from the early

1970s presented in Dahlhaus (1991), and buildings with low tolerance for total or differential settlement are typically piled. However, there are numerous public open space parks and sports ovals built over backfilled former quarry pit (and gully fill landfills) in Melbourne as well as commercial or public buildings (e.g. Altona Gates and Northcote Plaza shopping centres and Footscray Technical School), low-rise residential developments (Bouazza & Avelle, 2006) and medium to high rise buildings including the piled multi-level car park at the Tooronga Village and townhouse development in Cavanagh Street, Cheltenham. Provided that development is planned in recognition of the specific geotechnical challenges then development is technically feasible. However, where linear infrastructure such as roads and services will be constructed over uncontrolled fill materials there should be an expectation of ongoing ground settlement which may require ground improvement to reduce but not eliminate the magnitude of post-construction settlement, and needs to be communicated at an early stage to the developer and relevant planning authority (e.g. Council).

For former landfills, the age of the landfill and whether it is lined are key development considerations. Modern landfills (refer EPA Victoria, 2015) are typically constructed with low permeability side and base liners and have a surface capping of low permeability materials. These features can restrict their development potential as it may not be permitted to alter the capping or penetrate the base liner (for example) with pile foundations. However, older landfills (including older unlicensed landfills) typically do not have base or side liners. Where a capping layer is present above older landfills it is usually practical to reinstate the cap or otherwise provide satisfactory environmental provisions around pile penetrations. Note that mounding of the surface cap to help with surface water runoff, and restrictions on earthworks such as excavation into waste materials below the cap, can pose challenges for achieving surface grades conducive for development.

Environmental considerations are critical to the development of backfilled quarry pits but are not addressed in detail in this paper.

4.2 Site characterisation and settlement

The geotechnical characterisation of backfilled quarry pits can be challenging as there are usually few (if any) records of the backfill materials used and it can be challenging to identify the footprint of the former quarry pit once it is backfilled. Landfill materials and the cap also typically extend beyond the perimeter of the former quarry pit.

Key information sources to assist in the characterisation of backfilled quarry pits include historical aerial photographs, survey and quarry plans including photogrammetric mapping, as well as landfill licence documents where relevant and available, and information from more conventional geotechnical and geophysical investigation

techniques. Where landfill waste with a capping layer is present it is important to assess the thickness and nature of the cap (which may more accurately be called a layer of cover soil with minor or no waste rather than a formal engineered cap as would be constructed over a modern landfill).

Key challenges for geotechnical and geophysical investigation techniques include the following:

- Health and safety risks including the potential to encounter landfill gas and explosive/flammable atmospheres as well as contaminated soils, leachate or groundwater.
- Obstructions or materials such as wire can be encountered during drilling which can cause damage to the investigation equipment (e.g. broken drill rods) and delays to investigations whilst boreholes are relocated and redrilled.
- Where relevant, requirements to limit disturbance to the landfill cap and reinstate penetrations through the cap.
- Difficulty in interpreting geophysical results in variable backfill materials.

The amount and rate of settlement depends on a number of factors including the depth of the backfill materials and their composition (including any waste materials present), the age of the backfill and the type of plant and methodology used to initially place and compact the backfill materials. Furthermore, if additional fill is placed over existing uncontrolled backfill materials, additional settlement will occur.

In general terms, the settlement that occurs comprises primary settlement in response to the application of a new load (e.g. the weight of additional fill), and secondary or creep settlement that occurs in the absence of any new loads (e.g. due to the decomposition of putrescible materials in the landfill and consolidation of the fill under its self-weight). Secondary or creep settlement is usually the main concern, assuming that ground improvement works such as fill surcharging and/or dynamic compaction are undertaken prior to the commencement of building works to address issues associated with primary settlement (i.e. the settlement behaviour of the landfill is monitored and assessed during ground improvement works, which are designed to 'build-out' the additional primary settlement before construction of permanent structures commences – this process may take one to two years or longer).

Ground improvement works will not eliminate secondary settlement. However, monitoring of landfill settlement behaviour during preparatory works provides the opportunity to assess the rate of secondary settlement. Ground improvement works can also assist in building a 'bridging layer' of relatively well-compacted fill near the ground surface which can help reduce the potential for differential settlement due to secondary settlement in the underlying landfill materials.

The magnitude and rate of primary and creep settlement is difficult to predict in advance and usually requires the performance of a monitored field

trial (e.g. trial embankment, refer Figure 2). Yuen & Styles (2000) provide some useful settlement data for landfill materials placed in a former sand quarry in the southeast suburbs of Melbourne.



Figure 2. An example of a surcharge trial pad constructed to help assess settlement rates

4.3 Common issues for development of backfilled quarry pits

Common issues observed in the development of backfilled former quarry pits include the following:

- Health and safety risks, e.g. the potential to encounter landfill gases and contaminated soils, leachate and groundwater.
- Inability to identify and classify uncontrolled backfill materials, or a lack of communication/knowledge transfer about the presence of uncontrolled fill leading to inappropriate development. Ideally, a single developer should oversee development from start to finish to manage risks rather than selling allotments to a third party to develop. If uncontrolled backfill materials are present, it is important they are identified as such in geotechnical investigations. This may require review of historical aerial photographs or other information sources. It is also important to recognise that placing a surface layer or layers of controlled fill does not allow for reclassification of a site in accordance with AS2870 (2011).
- Obstructions to piling or other construction activities may be present within the backfill materials. The presence of steep former pit wall slopes is also important as piling near these slopes can result in significant pile breakage or loss of verticality. Trial piling works can assist.
- Where piling in fill subject to ongoing settlement is required, due consideration of downdrag/negative friction loads is required.
- The ground support conditions and requirements for working platform construction need to be carefully considered.
- Unrealistic expectations about the benefits of ground improvement, i.e. ground improvement work such as surcharging, deep dynamic compaction, impact rolling.
- Difficulties with maintaining the integrity of monitoring equipment during trials, e.g. damage to settlement monitoring plates during trial embankment construction, or difficulty identifying a stable reference point for survey monitoring on large landfill sites.

5 OTHER CONSIDERATIONS

This paper focuses on the commonly encountered situation(s) where the quarry and/or landfill activities have made no prior plans for end of life redevelopment. Planning for future redevelopment whilst quarrying is still underway (or has not yet commenced) presents significant opportunities for sustainable and more cost-effective redevelopment at the end of the quarry or landfill life.

6 CONCLUSION

The redevelopment of former quarry and landfill brownfields sites offers benefits for community amenity and the sustainable reuse of geo-materials. Given the complexity of these sites, the geotechnical considerations as discussed in this paper are critical to the early development of a site and development specific strategy for redevelopment. It may not be practical to rectify poorly planned or poorly performed development works (e.g. excessive settlement of deep fill) and such adverse outcomes would increase the difficulty in obtaining regulatory approval for new projects.

Communicating the key geotechnical issues and the strategy to address risks to stakeholders including the developer and relevant regulatory authorities such as Council is also critical to progressing this type of redevelopment and providing regulators with confidence that redevelopment is feasible.

7 ACKNOWLEDGEMENTS

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SESSION 3
RECYCLED
MATERIALS



Keynote Address

Recent advances in the usage of recycled materials in transportation geotechnics

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ABSTRACT

Priority waste materials currently generated in Australia include construction wastes, demolition wastes, glass fines, waste tyres, plastics, industrial wastes and organic wastes. The increase in generation of these wastes have led to significant research over the past decade on the reuse of recycled waste materials in geotechnical engineering applications. An estimated 7.9 Mt of wastes, which accounts for 36% of Australia's current annual landfilled waste, have the potential to be diverted into civil engineering applications, such as for the construction of roads, railways and land reclamation projects. Recycled materials have been evaluated in the laboratory and new specifications successfully developed, to incorporate their usage in pavement geotechnology and ground improvement applications. Recycled materials are increasingly being used in unbound and stabilised pavement applications. In addition, industrial wastes such as fly ash and slag have also been evaluated in recent years as alternative binders to cement in pavement and ground improvement applications. This paper discusses recent advances in the usage of recycled materials in transportation geotechnics, with reference to case studies of recycled materials usage in Australian projects. Ground improvement projects, comprising of the installation of ground inclusions in waste materials, in an international railway and an airport land reclamation project are also discussed.

Keywords: sustainable geotechnics; pavement base; ground improvement; railways; recycled materials; waste.

1 INTRODUCTION

An estimated 7.9 Mt of wastes, which accounts for 36% of Australia's current annual landfilled waste, have the potential to be diverted into geotechnical engineering applications, including roads, railways and land reclamation projects. The recycling and reuse of solid wastes for civil engineering projects has become significant in recent years, given the shortages of natural resources and the increasing cost of landfills.

Recycled aggregates, such as construction and demolition (C&D) wastes have in recent years become a mainstream pavement base/subbase material in road construction projects (Hoyos et al., 2011, Puppala et al., 2011, Arulrajah et al., 2018, Yaghoubi et al., 2018, Perera et al., 2019). The reuse of C&D materials in pavement geotechnics projects have reduced the demand for quarry materials and diverted traditional waste materials from landfills (Disfani et al., 2011, Hoyos et al., 2011, Vieira et al., 2020). In addition to C&D materials, other recycled aggregates such as steel slags have also made significant inroads into Australian road projects (Maghool et al., 2019).

Ground improvement projects are increasingly being undertaken on waste tailings and mining ponds. These project involve land reclamation and ground improvement works inclusive of prefabricated vertical drains (PVDs), stone columns and deep soil mixing. Dredging spoils have also been used as a sustainable land reclamation fill material.

This paper discusses some recent advances in the usage of recycled materials in transportation geotechnics projects in Australia and internationally.

Case studies of geotechnical engineering projects in pavements, airport and railway construction where recycled materials have been implemented will also be discussed.

2 RECYCLED MATERIALS IN PAVEMENT BASES

2.1 Unbound recycled C&D aggregates

The main components of C&D materials include; Recycled Concrete Aggregate (RCA), Crushed Brick (CB), Reclaimed Asphalt Pavement (RAP), Waste Rock (WR) and Recycled Glass (RG). Recycled CB, RCA, WR and RAP used in pavement geotechnics projects in Australia have a maximum particle size of 20 mm. RG has a maximum particle size of 4.75 mm (Disfani et al., 2011).

For each recycled material or blend of recycled materials, a suite of extensive geotechnical tests are undertaken to ensure the properties of the recycled materials meet the requirement of road authorities. Characterisation tests undertaken include: particle size distribution, particle density, modified compaction, organic content, pH, hydraulic conductivity, flakiness index, Los Angeles abrasion loss, California bearing ratio and repeat load triaxial (RLT) tests. In addition to geotechnical testing, leachate analyses are undertaken on C&D materials, to ensure the materials are below the threshold required for hazardous waste. The particle size distributions of these C&D materials are shown in Figure 1.

The RLT test is a specialised testing method to determine the resilient modulus and permanent deformation of unbound recycled materials. The

RLT test consists of two phases, permanent strain testing followed by resilient modulus testing. Permanent strain testing phase consists of three or four stages, each undertaken at different deviator stresses and a constant confining stress. The resilient modulus testing phase consists of 66 loading stages with 200 repetitions. Figure 2 shows the permanent deformation results of C&D materials while Figure 3 shows the resilient modulus results.

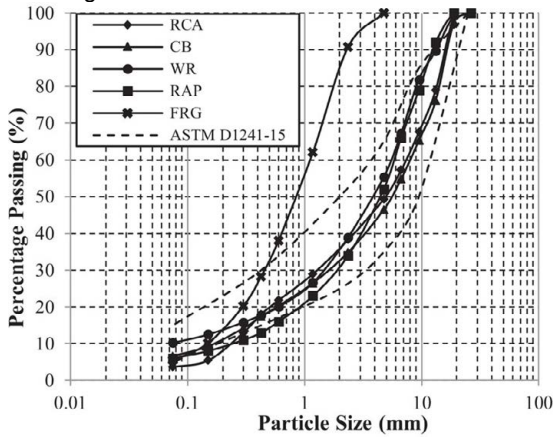


Figure 1. PSD curves of C&D materials (Arulrajah et al., 2013a)

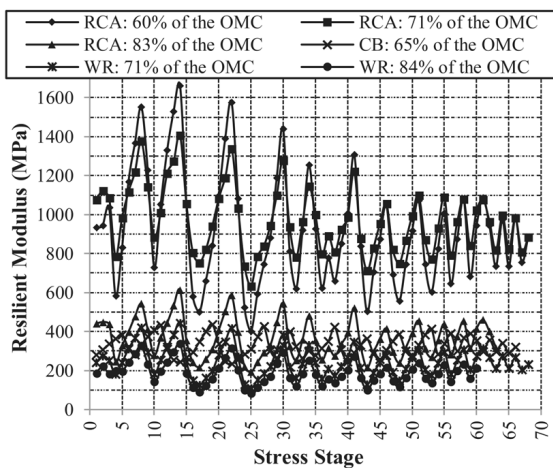


Figure 2. Permanent strain test results for C&D materials (Arulrajah et al., 2013a)

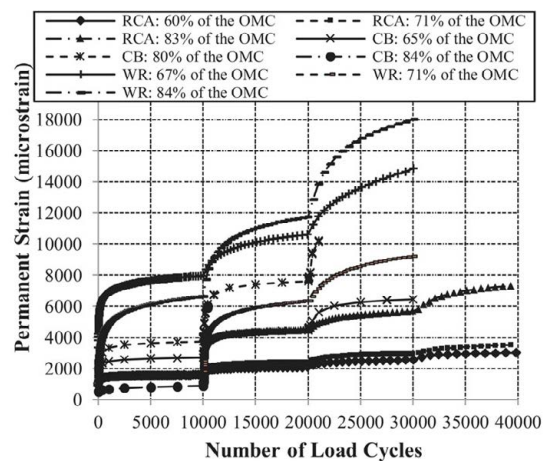


Figure 3. Resilient modulus test results for C&D materials (Arulrajah et al., 2013a)

2.2 Chemically stabilised wastes

Chemical stabilisation is a ground improvement method that is used to improve the strength of soils and aggregates in transportation geotechnics projects. Cement and lime are traditional binders used for the chemical stabilisation of soils and aggregates. In recent years, alternative binders comprising of waste by-products such as with alkali-activated geopolymers have been evaluated for the chemical stabilisation of geomaterials.

2.2.1 Cement stabilised wastes

Research has been undertaken to evaluate the performance of recycled materials in blends with other supplementary recycled products. Figure 4 shows the UCS values for RCA/RG blends, indicating the values met the local road authority minimum requirement of 3.5 MPa, for a minimum of 7 day curing. Further strength increase is evident after 28 days of curing, as compared to after 7 days of curing.

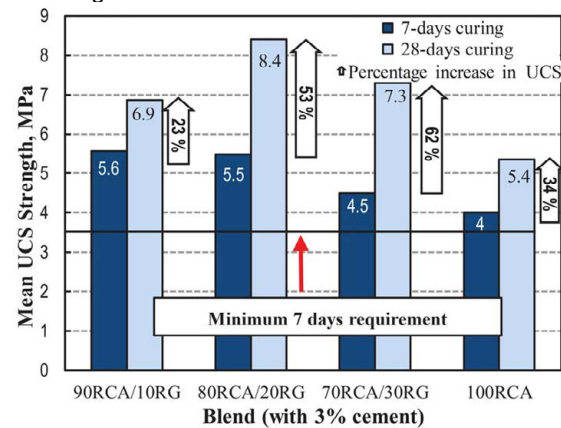


Figure 4. UCS results of cement stabilised RCA/RG blends (Arulrajah et al., 2015a)

RLT tests are often undertaken on cement stabilised materials to ascertain the performance under simulated traffic loading conditions. Figure 5 shows typical permanent deformation results for a cement stabilised RCA/RG blends.

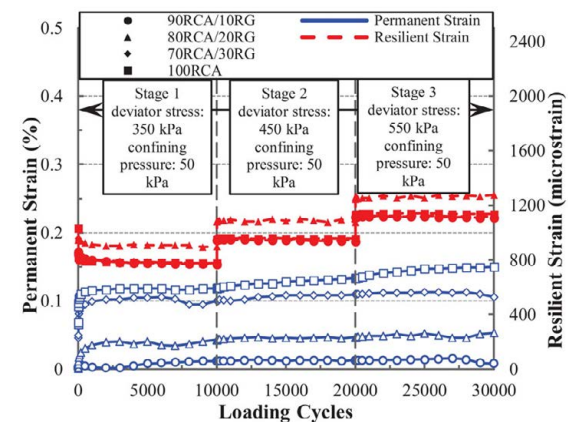


Figure 5. Permanent deformation results of cement stabilised RCA/RG blends (Arulrajah et al., 2015a)

Figure 6 shows typical resilient modulus results for an RCA/RG blend. The figures indicate that an increase in the RG content in RCA would result in lower resilient modulus and higher permanent strains.

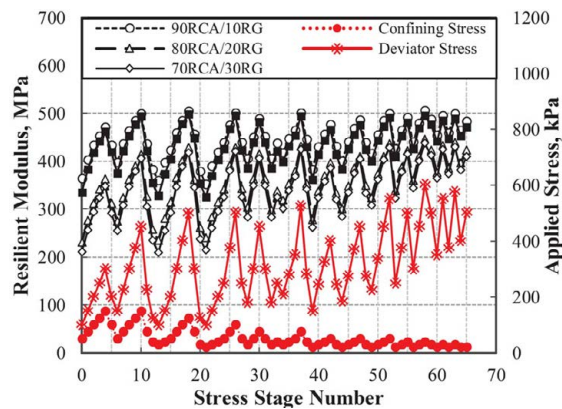


Figure 6. Resilient modulus results of cement stabilized RCA/RG blends (Arulrajah et al., 2015a)

Modulus of rupture and fatigue life for recycled products are determined using flexural beam tests. Figure 7 shows the modulus of rupture and flexural modulus for a cement-stabilised RCA/RG blends, indicating these blends are suitable for stabilised pavement bases/subbases. Evidently, the higher the RG content, the lower the modulus of rupture, flexural modulus and corresponding design modulus. The results of the flexural beam tests were compared to that of cement stabilised quarry crushed rock products, to ensure compliance with road authority specifications. Research on cement stabilised recycled products has indicated that the water/cement ratio needs to be maintained at 3.5 to ensure consistent testing outcomes.

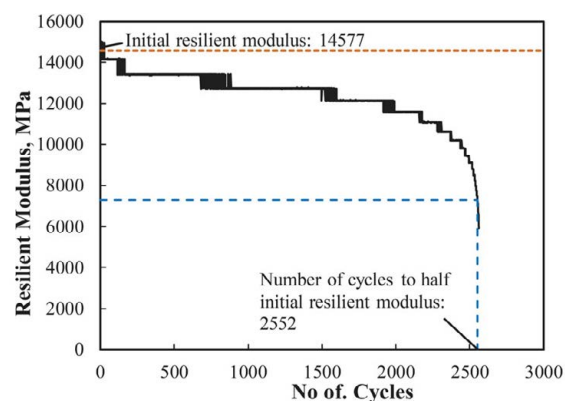


Figure 7. Fatigue test results of cement stabilised 70RCA/30RG blend (Arulrajah et al., 2015a)

2.2.2 Geopolymer stabilised wastes

Geopolymers have been researched as an alternative binder for ground improvement and pavement stabilisation works. Significant research has been undertaken on geopolymers from a laboratory testing and physical modelling perspective.

Geopolymers comprise of two prime components, being a precursor and an alkali activator. The precursor typically comprises of a material that is in the amorphous phase, such as fly ash (FA) or ground granulated blast furnace slag (S). The multi-compound activator on the other hand typically comprises of a sodium hydroxide and sodium silicate solution.

Other forms of precursors that have been used in recent years include calcium carbide residue, lime kiln dust, cement kiln dust, rice husk ash and bagasse. Potassium hydroxide and potassium silicate solution have also been explored as alkali activators for the stabilisation of demolition wastes.

Geopolymers have been evaluated for stabilisation of demolition wastes for road subgrades, subbases and bases (Mohammadinia et al., 2019, Cristelo et al., 2018). The effect of various temperatures and alkali contents have been studied to optimise the performance of geopolymers. Figure 8 shows UCS testing results from demolition wastes stabilised using potassium-based alkalis in combination with FA and S precursors.

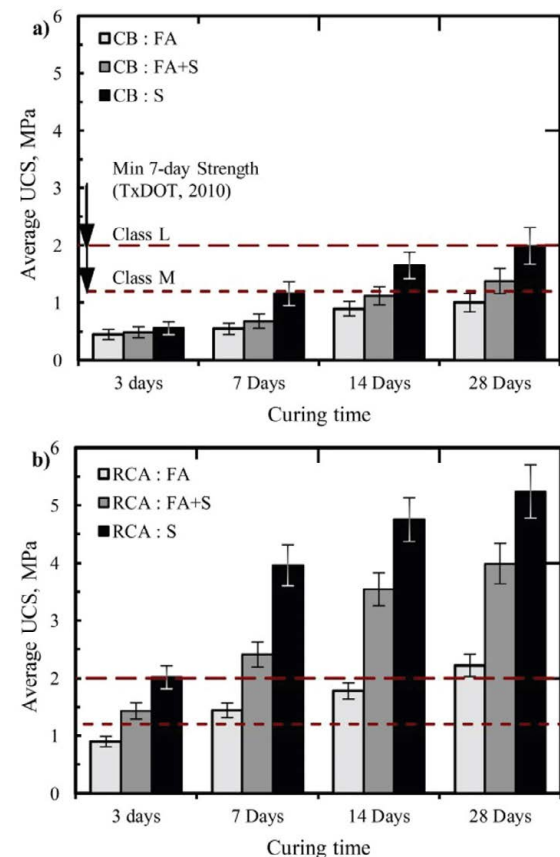


Figure 8. UCS of C&D wastes stabilised with potassium-based geopolymers: a) CB and b) RCA. (Mohammadinia et al., 2019)

Geopolymers have also been studied for the stabilisation of marine soils, for applications such as deep soil mixing. Figure 9 shows UCS testing results on a marine soil using geopolymers. Figure 10 shows the effect of temperature on the geopolymer stabilisation of a marine soil.

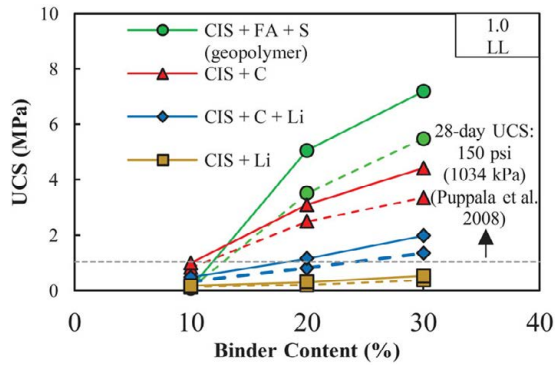


Figure 9. UCS of Coode Island Silt stabilised with geopolymers (Arulrajah et al., 2018)

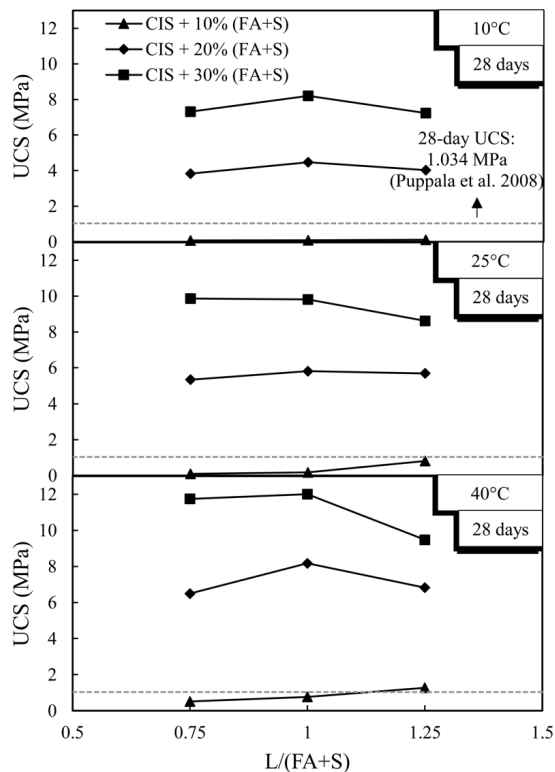


Figure 10. Effect of temperature on a geopolymer stabilised marine soil (Yaghoubi et al., 2019)

2.3 Field case studies

Following the completion of laboratory evaluation works, several field trial projects have been implemented in the state of Victoria, Australia.

2.3.1 C&D materials in bases/subbases

Nine sections of unbound granular base pavements, comprising of up to 30% RG in blends with RCA and WR, were constructed on the main haul road at a recycling site in the state of Victoria, Australia.

The 200 mm thick pavement subbase layer was constructed with RAP. Figure 11 presents a site photo showing the pavement subbase during placement of the RAP layer. The 200 mm thick pavement base layer was constructed with RG/RCA or RG/WR blends in 7 sections and with RCA and

WR for the 2 remaining control sections. Four sections were constructed with RCA with up to 30% RG content. Another three sections were constructed with WR with up to 30% content of RG. Figure 12 shows the laying of the RG base layer.

Field testing were undertaken after placement of the subbase and base layers with a Nuclear Density Gauge and Clegg Hammer. Figure 13 presents the Clegg Hammer results for the pavement base sections. The Clegg Hammer results met the specified minimum soaked field CBR of 80%. Figure 14 shows the compaction of the asphalt surface for the completed road.



Figure 11. Construction of RAP subbase (Arulrajah et al., 2014)



Figure 12. Construction of RG/RCA base (Arulrajah et al., 2014)

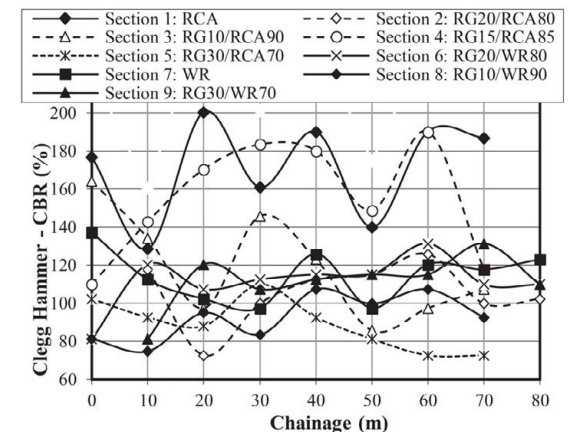


Figure 13. Clegg Hammer test results for the RG blends in pavement base (Arulrajah et al., 2014)



Figure 14. Construction of RAP subbase (Arulrajah et al., 2014)

2.3.2 Recycled glass in footpaths

An asphalt footpath for shared use by pedestrians and cyclists was constructed at the suburb of Manningham, Melbourne. The shared path comprised of a base layer of nominal 100 mm thickness, overlying a subgrade with a design soaked CBR greater than 3%. The base layer was overlaid by a 30 mm thick asphalt cover.

The asphalt footpath was constructed in three sections, with three different material blends of RG/WR in the footpath base layer. Three trial sections were constructed with 15% RG (RG15/WR85), 30% RG (RG30/WR70) and a control section comprising 100% WR.

Figure 15 shows the footpath sections after completion of field compaction of the base layer. Clegg Hammer test results as shown in Figure 16 were analysed to determine CBR values of the various footpath sections as well as to determine the strength ratios based on a required minimum soaked field CBR of 28% after field compaction.

3 GROUND IMPROVEMENT OF MINE TAILING SPOILS FOR A HIGH-SPEED RAILWAY PROJECT

A high-speed 110 km length railway project for trains of speeds of up to 160 km/h was constructed between Rawang and Bidor in Peninsular Malaysia. The railway project traversed a number of abandoned tin mine tailing ponds which had been disused since the end of the global tin mining boom in the late 1980s. Figure 17 shows the location of the railway project site.

The ground improvement methods adopted in the project included vibro-replacement with stone columns and dry deep soil mixing with cement columns. Ground improvement with deep inclusion techniques was required to ensure adequate performance of the embankments in terms of settlement and slope stability as well as completion of the project within the required project duration. The railway embankments in the project had heights ranging from 1 to 12 m. The soils encountered on the project site were highly variable mixtures of very soft silts and clays, as well as loose sands to depths of up to 30 m.



Figure 15. Footpath base sections with RG/WR blends (Arulrajah et al., 2013b)

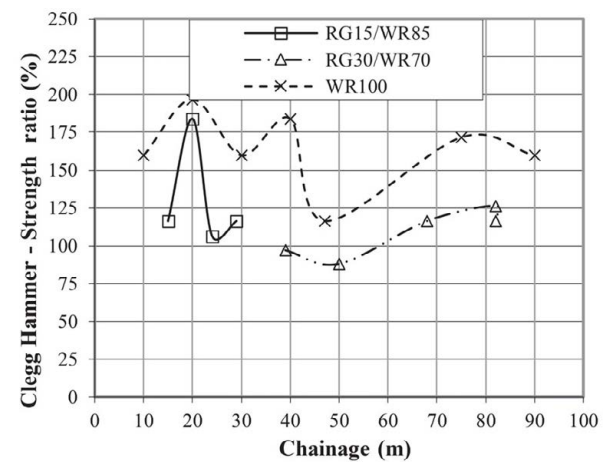


Figure 16. Clegg hammer results for footpath bases with RG/WR (Arulrajah et al., 2013b)

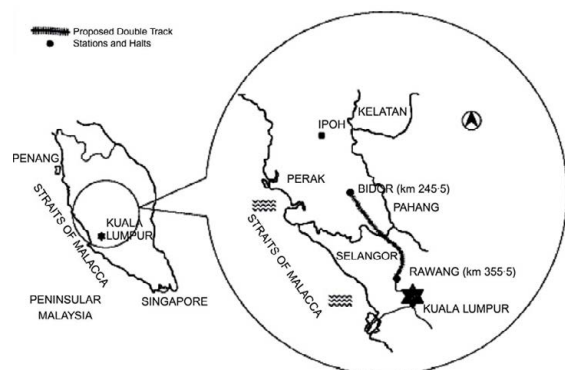


Figure 17. Location of the railway project site in Peninsular Malaysia (Arulrajah et al., 2009)

3.1 Stone columns

Vibro-replacement with stone columns is a subsoil improvement method in which large-sized columns of coarse backfill material are installed in the soil by means of special depth vibrators. The stone columns and the intervening soils form an integrated foundation support system having low compressibility and improved load-bearing capacity.

Vibro-replacement with stone columns allows for the treatment of a wide range of soils, from soft clays to loose sands, by forming reinforcing elements of low compressibility and high shear strength. In addition

to improving strength and deformation properties, stone columns densify in situ soil, rapidly drain the generated excess pore water pressures, accelerate consolidation and minimise post-construction settlement (Tai et al., 2020, Basack et al., 2017, Indraratna et al., 2015, Gu et al., 2017a, Gu et al., 2017b).

Figure 18 shows a schematic of the vibro-replacement with stone column treatment scheme. Figure 19 presents typical results from a settlement plate installed on the project site. Figure 20 presents an Asaoka plot for a typical settlement plate, which indicates that the degree of consolidation of the improved ground at the location was 94%.

3.2 DEEP SOIL MIXING

Dry deep soil mixing (DSM) is a form of soil improvement involving mechanical mixing of in situ soft and weak soils with a cementitious compound such as lime, cement or a combination of both in different proportions. The mixture is often referred to as the binder. The binder is injected into the soil in a dry form. The moisture in the soil is utilised for the binding process, resulting in an improved soil with higher shear strength and lower compressibility (Yi et al., 2018, Chai et al., 2015, Puppala and Pedarla, 2017, Nguyen et al., 2019).

Ground improvement by means of DSM allows for the treatment of a wide range of soils, by forming stronger reinforcing elements of low compressibility and high shear strength. Cement was used as the binding agent in the project, consisting of standard Portland type, grain sizes 0–0.01 mm, and with approximately 65% of activated CaO. The amount of binder is usually in the range 100–150 kg/m³ of soil. The final result of the deep soil mixing process is a soil mass in the shape of a cylindrical column with improved deformation and shear resistance characteristics. Figure 21 presents a schematic diagram of dry deep soil mixing treatment scheme.

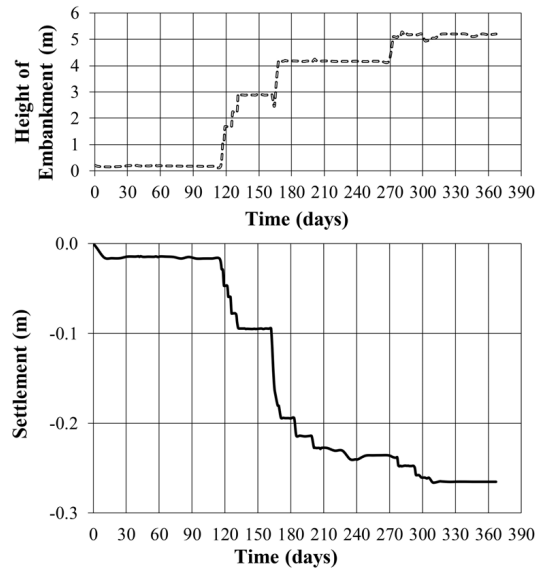


Figure 19. Plot showing the result of a settlement plate (Arulrajah et al., 2015b)

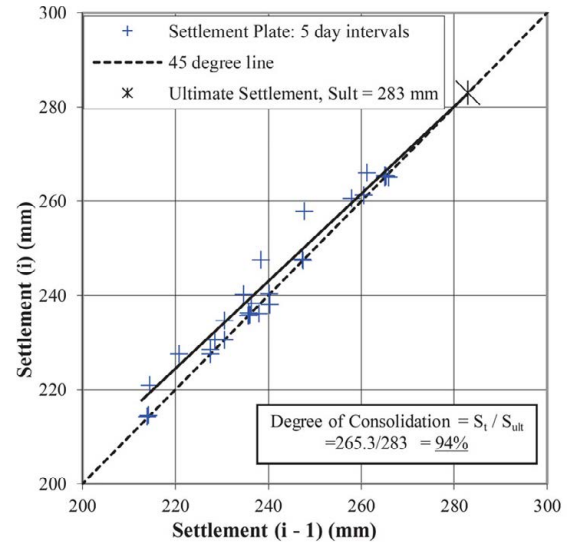


Figure 20. Asaoka plot for a typical settlement plate (Arulrajah et al., 2015b)

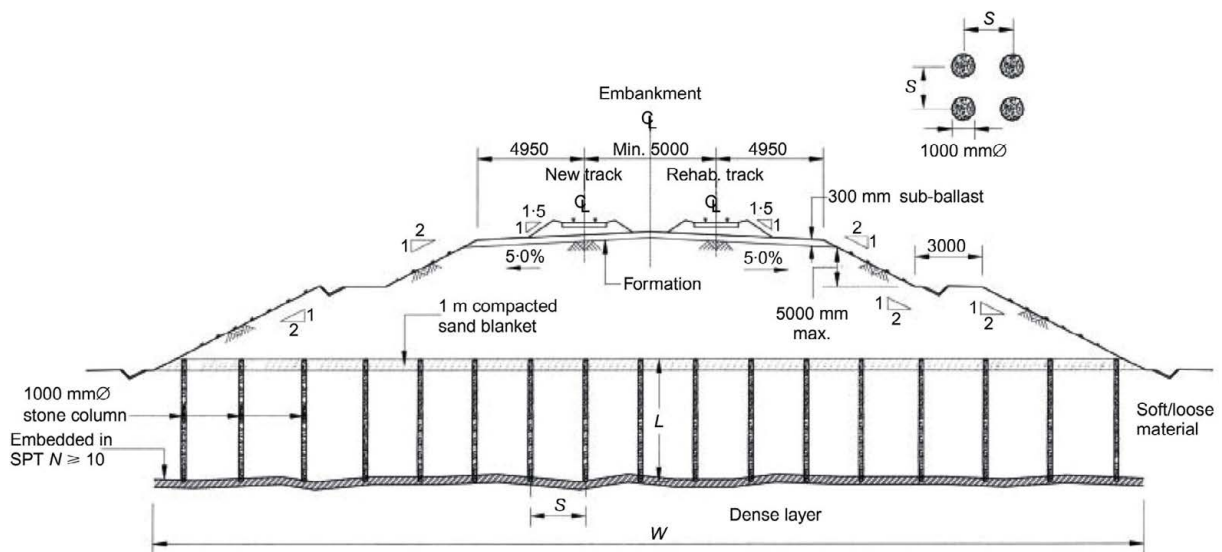


Figure 18. Schematic of the vibro-replacement with stone column treatment scheme (Arulrajah et al., 2009)

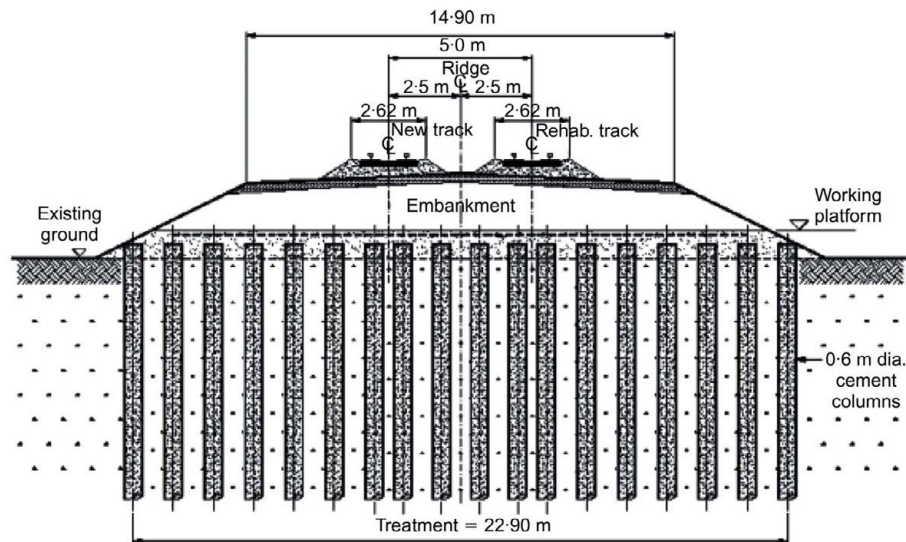


Figure 21. Schematic of dry deep soil mixing treatment scheme (Arulrajah et al., 2015b)

4 AIRPORT LAND RECLAMATION PROJECT USING DREDGED SPOILS

The Changi East reclamation project was carried out between 1991 and 2005 in five phases to create 2000 hectares of land for the expansion of the Changi International Airport and other infrastructure developments in Singapore. The water depths in the reclaimed area ranged from 5 to 15 m. The project involved hydraulic placement of 272 million m³ of sand in seawater up to 15 m deep (Bo et al., 2018).

The majority of the land reclamation area was underlain by a highly compressible layer of Singapore marine clay, up to 50 m thick. The upper and lower marine clay were highly compressible and had a high water content. An intermediate stiff clay layer was present between the upper and lower marine clay layers at parts of the project site.

The ground improvement technique used to treat the underlying marine clay was with prefabricated vertical drains (PVDs) with surcharge. The total area of the soil improvement works was approximately 1200 ha. The location of the project site is shown in Figure 22.

Due to limitations with the dumping of dredged spoils outside the land reclamation area, the dredged spoils were deposited within the land reclamation site and subsequently treated with PVDs and surcharge. The project also included the land reclamation and ground improvement of a former sand mining tailing pond (locally termed as a Silt Pond), due to sand extraction works for an earlier phase of the Changi airport reclamation works.

4.1 Prefabricated vertical drains with surcharge

Approximately 170 million linear metres of PVDs together with surcharge up to 8 m were used to consolidate the seabed soft marine clay and improve its geotechnical engineering properties

The PVDs were installed when the sand fill reached a level slightly above the high tide. A fill surcharge 8 to 12 m high was then applied. The fill surcharge was chosen based on the anticipated maximum future loads to be applied.

The design specification for the runway in the project was that a degree of consolidation of 90% should be achieved in 18 months, after which surcharge could be removed. Figure 23 shows typical settlement and pore pressure measurements during the project

4.2 Deep sand compaction

The hydraulically placed sand fill was generally in a loose state and deep sand compaction techniques were used to densify the sand fill. Three deep compaction methods were deployed: dynamic compaction using heavy pounders, vibroflotation and Muller resonance compaction. The compaction works had to meet specification performance requirements, based on cone resistance values.

Figure 24 shows the layout of deep compaction in the land reclamation project. Figure 25 shows cone resistance measurements at a dynamic compaction location while Figure 26 shows cone resistance at a vibroflotation location.

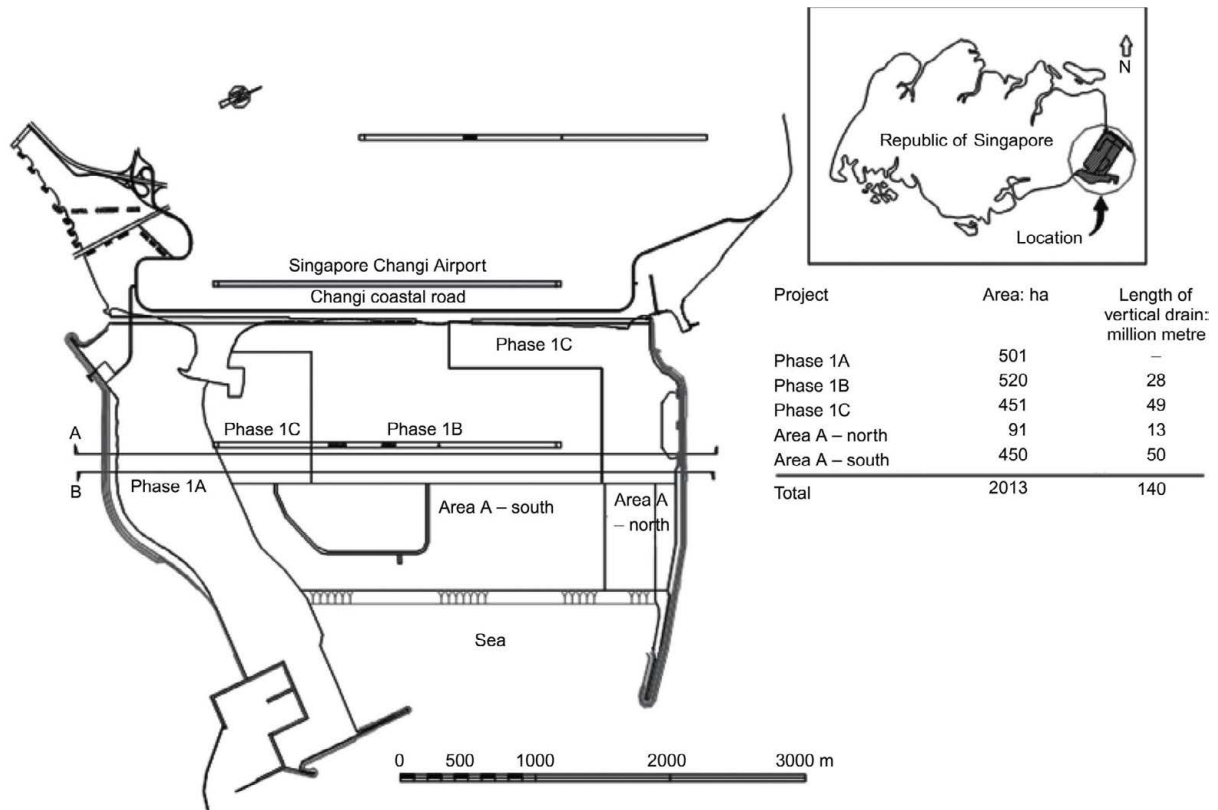


Figure 22. Location and site plan of the project (Bo et al., 2018)

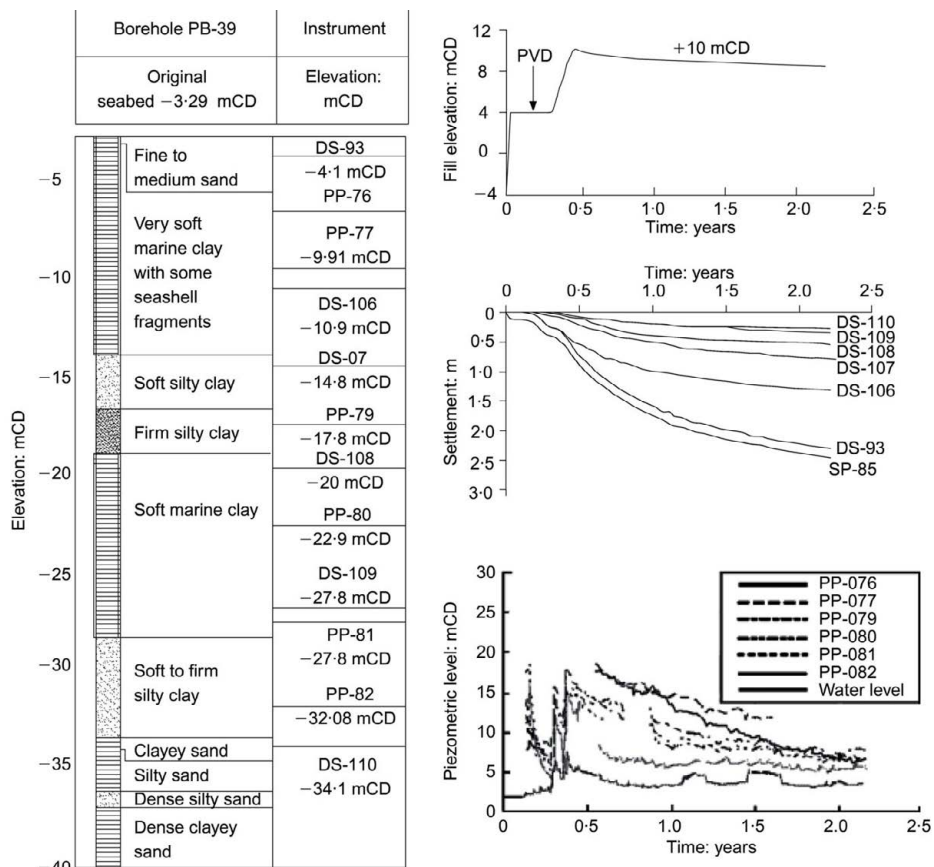


Figure 23. Settlement and pore pressure monitored during reclamation (Chu et al., 2009)

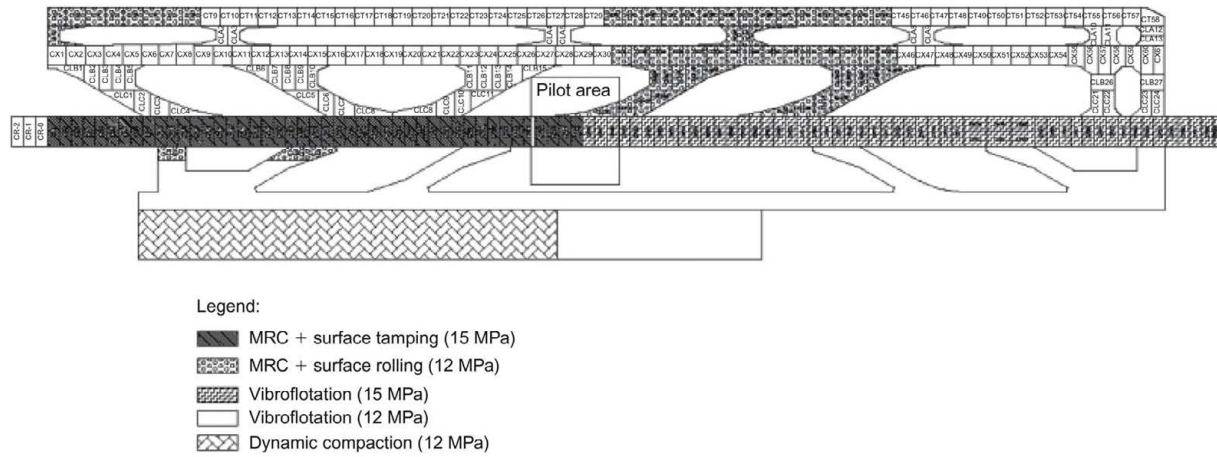


Figure 24. Layout of deep compaction works in the Changi airport land reclamation project (Bo et al., 2009)

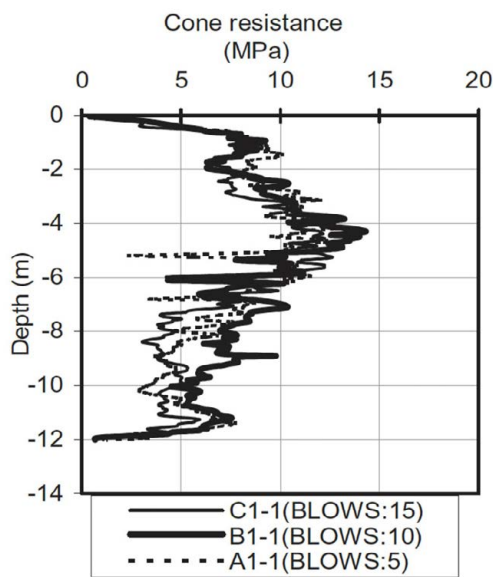


Figure 25. Cone resistance measurement at a dynamic compaction location (Bo et al., 2015)

5 CONCLUSIONS

This paper reports on the usage of recycled waste materials in several geotechnical engineering projects. The characterisation and implementation of recycled geomaterials for these civil infrastructure projects has been discussed. Several unique case studies have been described which describe the usage of recycled and waste materials in several national and international road, railway and airport projects.

Research on C&D materials for pavement bases/subbases in Australia has been undertaken over the past decade, resulting in these recycled materials becoming a mainstream road construction material. Significant quantities of C&D materials are currently being used in pavement and footpath projects in Australia and internationally. New research projects are presently looking at the usage of these materials in railway subballast/capping layers.

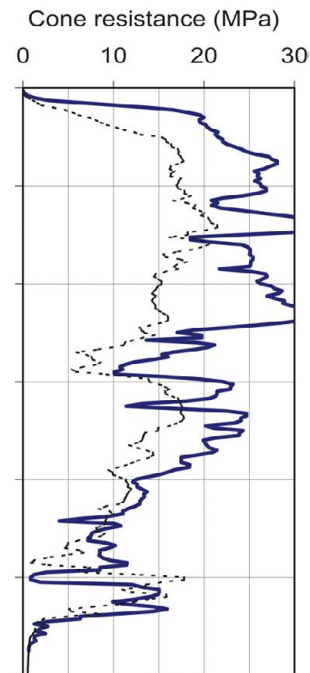


Figure 26. Cone resistance measurement at a dynamic compaction location (Bo et al., 2015)

Ground improvement techniques adopted for an international high-speed railway and an airport land reclamation projects using waste materials have also been described. In particular the usage of the PVDs, stone columns and deep soil mixing have been discussed. The ground improvement method to be used for the treatment of waste materials is dependent on various factors such as type of material, height of embankment and thickness of soft or loose deposits.

The usage of recycled waste materials in transportation geotechnics applications will result in a lower carbon footprint for civil engineering infrastructures. Recycled waste materials have in recent years also been found to provide cost and technical benefits. A sustainable engineering approach will assist to divert 7.9 Mt of wastes annually from Australian landfills and into future transportation geotechnics projects.

6 ACKNOWLEDGEMENTS

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Blends of recycled materials as sustainable alternatives for backfilling sewer trenches

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ABSTRACT

In this project, the suitability of four blends of recycled materials comprising different proportions of recycled glass, plastic, and tyre aggregates as alternative backfilling materials for deep excavated trenches was investigated. This paper presents results of an extensive testing program carried out for selecting two most appropriate blends for backfilling of trenches located in non-trafficable areas. These blends will be used for construction and instrumentation of trial sites for deformation monitoring over 12 months. Physical properties such as particle size distribution, maximum and minimum density, compaction properties, and field capacity of the blends were determined. Further, an application-specific geotechnical testing methodology was developed. This included determination of the dry density achieved using a proposed sand-raining technique (SRT) to simulate the real-life trench backfilling procedure and determination of the compressibility of the blends using a modified oedometer test. The SRT results showed that the obtained dry density (DD) increased as moisture content (MC) and height of drop increased. Based on the relative density achieved through the SRT compaction and the compressibility properties, Blends 2 and 4 showed the most advantageous characteristics. This paper presents the developed testing procedures and discussions on the results leading to the selection of the two most suitable blends for the proposed application. The paper also makes discussions on the stress-strain conditions expected on site and potential downfalls of the proposed application. The outcomes of this research aim to promote sustainable geotechnical design and construction by improving the industry's confidence in utilising recycled materials.

Keywords: recycled aggregates, trench backfill, sand-rain technique, compressibility, field compaction.

1 INTRODUCTION

The worldwide movement toward sustainability encourages continuous and persistent studies on new strategies of recycling and reusing waste material in a range of applications (Lindsey 2011). Since civil engineering construction projects have a significant demand for construction materials, they are a common destination for the reuse of recycled aggregates. Various studies have shown the suitability of using recycled materials, in particular recycled plastic, glass and tyre in civil engineering applications (Fauzi et al. 2016; Imteaz et al. 2012; Imteaz et al. 2018; Kamaruddin et al. 2017).

Sewer infrastructures backfilled with expansive clay or large soil clods result in poor backfill compaction during the variable seasonal cycles. Heavy winter rains expose these conditions and result in unexpected ground movements and subsequent failures, especially in rigid sewer pipes made of concrete. This issue is aggravated in narrow trenches for small diameter sewer pipes that are deeper than 1.5m where the quality control of the backfilled trenches can be challenging, if feasible, due to safety limitations restricting the field staff entering the trench for geotechnical testing. Temporary shoring or benched excavation of sidewalls are measures that can accommodate safe presence of testing staff in the trench for quality control. However, site conditions or space limitations may not always allow for such measures. Lack of proper quality control can lead to further settlements at the surface and affects both building structures and boundary fence lines. A potential solution is the application of blends of recycled

materials that have self-compacting characteristics, have small volume changes with moisture changes or require less compactive effort to reach the desired relative density and thus exhibit less settlement on the surface.

The current study aims to investigate the utilisation of recycled material blends as sewer trench backfill material in order to minimise the settlement of sewer backfill, in particular at non-trafficable manholes or sewage pit locations, which result in these structures protruding the surface, causing trip hazards as the backfill soils settle. The current project comprises application-specific laboratory testing programs, as well as the construction of a full-scale site trial for performance monitoring. This paper presents the geotechnical results and discussions to determine two mix designs of recycled material blends for the construction of the trial sites. The trial site is located in a greenfield area in the border of Tarneit and Truganina, Victoria. The Geological Survey of Victoria 1:63,360 Melbourne map sheet indicates that the site is underlain by Quaternary Age Newer Volcanics, comprising residual clay overlaying variably weathered basalt rock.

To the best of the authors' knowledge, this is the first time an investigation of 100% recycled material blends for sewer trench backfilling is being undertaken, leading to the construction of a real-life full-scale site trial. The outcome of this research can lead to an increased market demand for recycled materials, in particular, recycled plastic, recycled glass and tyre derived aggregates and reduce the reluctance of end-users for investment in the new innovative solutions.

2 METHODOLOGY

2.1 Materials and proportions

Hand mixed blends of 100% recycled materials including recycled glass (RG), recycled plastic (RP) and tyre derived aggregates (TDA) were used. The materials were supplied from local recycling facilities in Victoria, Australia. The specific gravities of the recycled materials were obtained following the AS1289.3.5.1 (2006) procedure to be 2.48, 1.1 and 1.12 for RG, RP and TDA, respectively.

The strategy to determine a suitable blend was selecting blends having a spectrum of different percentages of recycled materials with gradation curves that fall within Class 4 upper and lower band gradation (VicRoads 2013) as recommended by MRWA (2013). Consequently, four blends with different recycled material contents were selected as presented in Table 1.

Figure 1 shows one of the proposed blends in a Perspex cell with a perforated base plate used for the determination of the field capacity.

capacity is the amount of moisture held in soil after excess water has drained.



Figure 1. Blend 1 in a Perspex cell

Table 1: RG, RP and TDA ratios in blends 1 to 4

Recycled materials	% by Mass			
	Blend 1	Blend 2	Blend 3	Blend 4
RG	77	77	69	84
RP	6	9	6	5
TDA	17	14	25	11

2.2 Standard Compaction

Standard Proctor compaction test was performed according to AS1289.5.1.1 (2017). This test was carried out to obtain the optimum moisture content (OMC) and maximum dry density (MDD) based on the traditional compaction method as a benchmark.

2.3 Sand-Rain Technique

The sand-rain technique (SRT) setup developed in this study is demonstrated in Figure 2. The system consists of five parts: beam with adjustable height, hanging rod, hopper, collection tank and sample collector. The cross-section of the hopper from the top is 200(L)×200(W) mm and the short height is 200mm while the long height is 450mm. The hopper is equipped with a lid which opens up to 50mm. The beam and hanging rod allow the hopper to move in horizontal and vertical directions, respectively. The mould with 150mm diameter and 180mm height is placed in a tank of 370(L)×470(W)×700(H) mm. The base of the mould has 2mm diameter holes (approximately 1200 holes/m²) to allow drainage. The position of the hopper was adjusted by moving the beam vertically to allow for various heights of drop (HD) of the materials. The HD varied from 500mm to 2000mm. The mould was placed in the collection tank to collect the materials dropped out of the mould. The mould was raised on a collar to allow water draining from the base.

The moisture content (MC) of the blend varied from 3% (as received MC) to 18%. The MC of 18% was selected as preparing wetter samples did not lead to considerably greater dry density. At high MCs (≥15%), after dropping the materials the blend was left in the mould for 10 minutes allowing the water to

drain from the base of the mould. The moisture at this state was "field capacity" at which no extra water was drained off the sample due to gravity. Finally, the post-test MC was obtained and the dry density was determined. This procedure was repeated at various MCs and HDs.

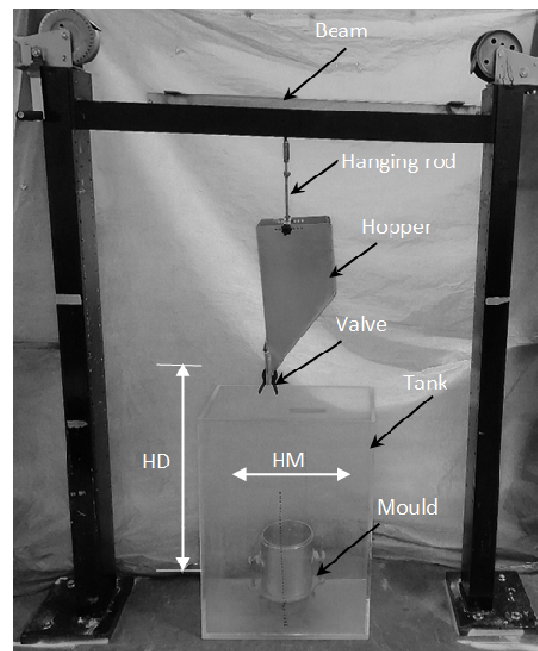


Figure 2. Sand-rain technique setup

2.4 Modified oedometer test

The compressibility of the blends was investigated following the AS1289.6.6.1 (1998) procedure with

modifications on the mould size and compaction of the sample. The inside walls of a mould with 150mm diameter and 120mm height were lubricated using silicon grease to reduce the errors due to friction. The base of the mould had 2mm through holes to allow drainage. A filter paper was used on the base and the sample was then placed into the mould in three layers to a total height of 75mm to provide a diameter to height ratio of 2, as recommended by AS1289.6.6.1. The oedometer test set up is presented in Figure 3.



Figure 3. Modified oedometer test setup

Several loading stages of 6, 12.5, 25, 50, 100, 200kPa were applied to the sample. At each loading step, the settlement was recorded and once no further settlement was achieved, the next load was applied. At the end of the test, the weight of the sample and the MC were measured to calculate the final density/void ratio. Also, the test data was collected to plot the axial strain-stress relationship. A similar procedure was undertaken for testing the compacted samples under saturated conditions. For the saturated samples, the mould was inundated in a water bath after a low surcharge of 1kPa was imposed on the sample.

3 RESULTS AND DISCUSSIONS

3.1 Particle size distribution

The PSD of the blends is presented in Figure 4 together with the upper and lower limits of Class 4 gradation based on VicRoads 2013. PSD curves show that the blends fall within the Class 4 gradation on the coarse side (>2.36 mm); however, they are slightly outside the lower limit on the fine side (<2.36 mm).

3.2 Standard Compaction

The standard Proctor OMC and MDD for Blends 1 to 4 were (8.5%, 1.34 t/m³), (9.5%, 1.36 t/m³), (11%, 1.25 t/m³) and (9%, 1.37 t/m³), respectively. Evidently, the OMC and MDD for Blends 1, 2, and 4 were close (9% and 1.36 t/m³ as an average). However, the MDD for Blend 3 was noticeably lower than those of other blends.

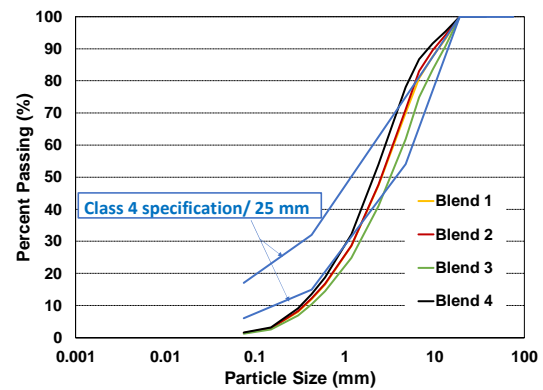


Figure 4. PSD of the four blends

3.3 Sand-rain technique

The relationship of dry density (DD) to post-test moisture content (MC) at different height of drops (500, 750, 1000, 1500 and 2000 mm) is plotted for Blends 1 to 4 in Figures 5 to 8, respectively. In these plots, the compaction curves achieved through standard compaction are also presented for comparison. Data labels show the initial (pre-compaction) MCs.

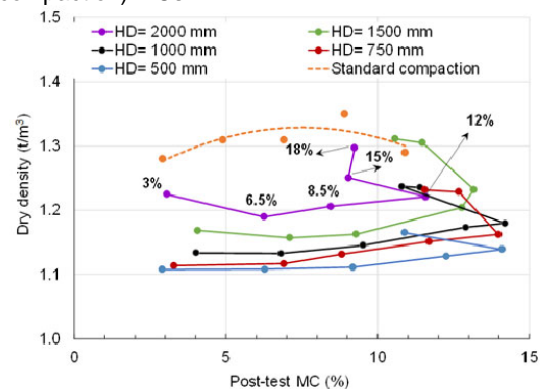


Figure 5. DD vs MC at various HD for Blend 1

Figures 5 to 8 show that the DD increased as MC increased at a constant HD. For initial MCs equal to or greater than 15% after dropping, the water drained from the sample and resulted in a post-test MC between 9% and 12.7%. Interestingly, this range aligns with the measured field capacity of the blends as presented in Table 2. As an example, in one of the trials, Blend 4 was prepared at MC of 18% and dropped from HD of 2000 mm. The blend was left for 10 minutes for draining until no further drainage occurred. The post-test MC was measured to be 9.7%. The MC of Blend 4 at its field capacity was measured to be 9%. For samples prepared at MC≥15%, the combined effect of higher lubrication and greater densification (lower voids) led to a greater drainage. This resulted in turning the compaction curve into the opposite direction and thus, two DDs for the same MC. The lower DD was corresponding to the lower initial MC with no drainage and the higher DD was corresponding to the higher initial MC which reduced through drainage to reach the post-test MC.

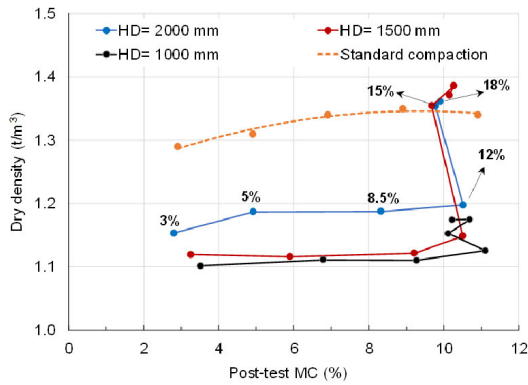


Figure 6. DD vs MC at various HD for Blend 2

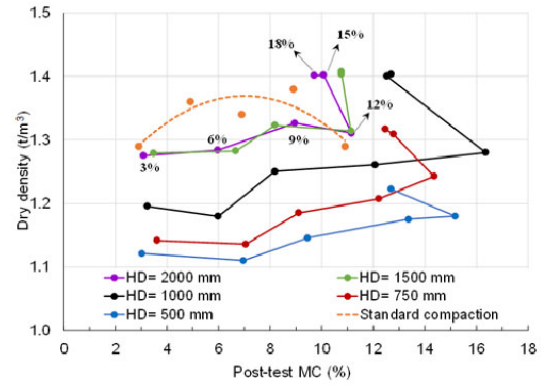


Figure 8. DD vs MC at various HD for Blend 4

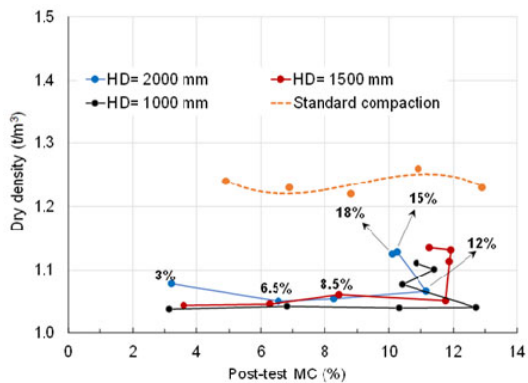


Figure 7. DD vs MC at various HD for Blend 3

Figures 5 to 8 also show that as HD increased, DD increased at a constant MC and consequently, it was expected that the post-compaction MC decreased as the height of drop increased. The increase in HD from 1500mm to 2000mm led to an increase in DD for blends with low MC. Once MC became close to 15% or higher, the change in DD became negligible. For Blend 4, the change in HD did not change the DD even for low MC ranges.

From the results, it may be concluded that the OMC and MDD of SRT compaction were achieved by dropping the blends from HD equivalent to 2000mm at high MCs ($\geq 15\%$). The OMC and MDD for Blends 1 to 4 were determined to be (18%, 1.3 t/m³), (15%, 1.35 t/m³), (15%, 1.13 t/m³) and (18%, 1.4 t/m³), respectively. Post-test MCs were 9.2, 9.7, 10.3 and 9.7 for Blends 1 to 4, respectively.

The dry densities obtained through the sand-rain technique for Blends 1, 2 and 4 (Table 2) show that a density comparable to MDD of standard Proctor can be achieved at a HD of 2000mm. The relative density with respect to max/min density test results (AS1289.5.5.1 2016) was greater than 80% for Blends 1, 2 and 4, being 82.5%, 85.5% and 83.5, respectively and the percentage of MDD obtained from the standard Proctor compaction were 97%, 99% and 102.2%, respectively (Table 2).

The relative density of Blend 3 was 52% which was significantly lower than the other blends. Therefore, Blend 3 was deemed unsuitable for construction of the full-scale site. The lower relative density of the backfilled material can lead to greater potential settlements after construction.

Table 2: Results of different compaction tests and field capacity test

	Blend	1	2	3	4
Max/Min Density	Maximum density AS1289.5.5.1	1.38	1.41	1.27	1.50
	Minimum density AS1289.5.5.1	1.02	1.08	1.01	1.05
Standard Proctor (SP)	Estimated MDD (t/m ³)	1.34	1.36	1.25	1.37
	Estimated OMC (%)	8.50	9.50	11.00	9.00
	Relative density	91.5	88	93.7	77.8
Sand-Raining technique (SRT)	Estimated MDD (t/m ³)	1.30	1.35	1.13	1.40
	Height of drop (mm)	2,000	2,000	2,000	2,000
	Target MC (%)	18.0	15.0	15.0	18.0
	Post-test MC (%)	9.2	9.7	10.2	9.7
	Relative density (%)	82.5	85.5	52	83.3
	Percentage of Field capacity (%)	97	99.3	90.4	102.2

3.4 Compressibility of the blends

A modified 1-D oedometer testing procedure as described in Section 2.4 was carried out to select two blends from Blends 1, 2 and 4 for construction of the trial site. The samples were prepared at the OMC and MDD achieved through the SRT compaction. Figure 9 shows that the total axial strain achieved in Blends 1, 2 and 4 were lower than that of Blend 1 under the loading steps mentioned in Section 2.5. Consequently, based on the oedometer test results,

Blends 2 and 4 were selected for the construction of the full-scale trial site.

3.5 Discussion on the construction MCs and potential segregation

Mixing the blends with MCs greater than 15% (about 6% greater than the field capacity of the blends) may result in excess water ingress into the natural ground and cause settlements or subsidence. On the other hand, densities achieved through SRT test show that

the blends may not achieve a satisfactory density if mixed with a MC close to their field capacity (Figures 5 to 8). Dropping Blends 2 and 4 at post-test MCs (9.7%), results in dry densities of 1.2 t/m³ and 1.33 t/m³, respectively (Figures 6 and 8).

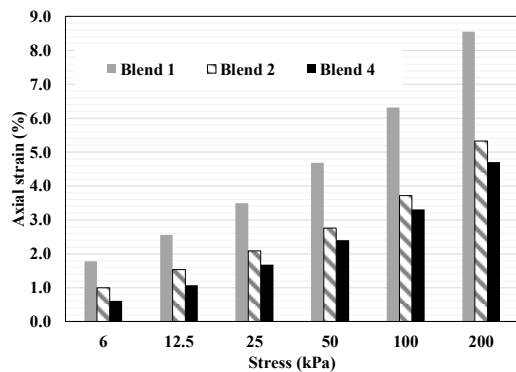


Figure 9. Axial strain-stress relationships for Blends 1, 2 and 4 at OMC and MDD

It should be noted that the density achieved through SRT compaction is the loosest state of the densified blends. During the backfilling process, the surcharge of the upper layers causes further densification of the bottom layers. In addition, the backfilled area will be further compacted using the conventional field compaction techniques such as excavator's compaction plate. In the case of excavated trenches of this project, the top 0.5m will be backfilled with conventional materials (site-won soils or class 4 aggregates). Figure 10 shows a schematic profile of the trench with three zones, being the embedment zone, recycled material backfill zone, and conventional backfill zone. Considering the discussions above, it is expected that the final density of the blends falls between the DD achieved at MC of 9.7% and those achieved at MC > 15% (Figures 6 and 8). Therefore, further compressibility assessment was carried out on samples prepared at MC of 9.7% and a density lower than the MDD achieved through SRT compaction.

Compressibility of Blend 2 was further assessed by preparing samples at (9.7%, 1.2 t/m³), (9.7%, 1.25 t/m³), (9.7%, 1.3 t/m³) to be compared with those of a sample at (9.7% and 1.35 t/m³). Also, Blend 4 was prepared at MC of 9.7%, and DD of 1.33 t/m³ to be compared with a sample prepared at MC of 9.7% and MDD of SRT compaction (1.4 t/m³).

For Blend 2, the change in axial strains obtained due to imposing a pressure up to 50kPa is minor for blends prepared at DD of 1.25 t/m³, 1.3 t/m³ and 1.35 t/m³. Assuming a maximum density of 1.41 t/m³ (AS1289.5.5.1) and a depth of 4m, the maximum surcharge at the bottom of the backfilled area is estimated to be 55kPa. The axial strain obtained by loading Blend 2 samples prepared at DD of 1.2 t/m³ was considerably higher than that of the other blends (Figure 11). As discussed before, the surcharge from the ordinary backfilled zone, together with additional compactive effort using the excavator is expected to increase the DD from 1.2 t/m³ to at least 1.25 t/m³. The axial strain shows a minor difference for Blend 2

samples prepared at the field capacity moisture content, and DDs of 1.25 to 1.35 t/m³.

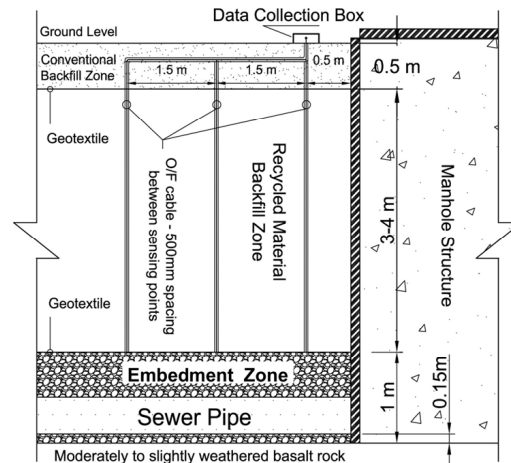


Figure 10. Schematic profile of a backfilled trench

Figure 11 shows that the change in axial strain between the two Blend 4 samples under a pressure less than 100kPa was minor. The difference in axial strain between the loosest and densest state of the compacted Blend 4 was significant when the blends were subjected to a pressure greater than 100 kPa. As the bottom of the recycled material backfilled zone is at a depth of about 4m and assuming the maximum density (AS1289.5.5.1 2016) of Blend 4 (1.50 t/m³) the maximum surcharge can be calculated to be 59kPa which is significantly lower than 100kPa. Consequently, despite achieving a lower density compared to the MDD of SRT compaction, it is expected that a negligible difference in axial strain is caused due to mixing at lower moisture content. It should be noted that due to the granular nature of the blends, a large proportion of the deformation occurs during construction and unlike clay, consolidation settlement is not expected.

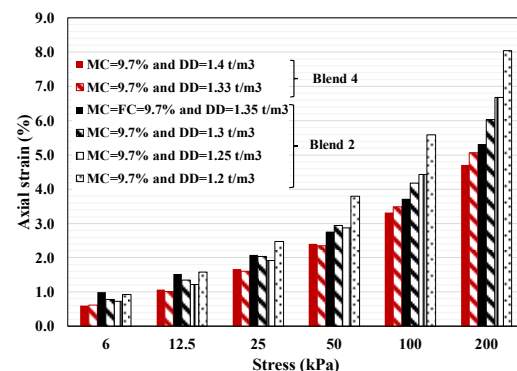


Figure 11. Axial strain-stress relationship for Blends 2 and 4 at post-test MC and DD ≤ MDD

It is anticipated that civil contractors will favour a load and dump technique similar to that used in normal backfill construction. Field trials will consider the influence of the drop height from the bucket, with elevated heights possibly required as the trench is filled to near the finished fill depth. Alternatives including an Aggie truck or volumetric mixer truck may be considered to control the height of drop. A potential concern during the backfilling procedure is the segregation of aggregates. According to the

studies of Tripathy et al. (2002) and Islam (2015), the OMC achieved in the standard compaction method is expected to result in a degree of saturation of 85%. The MC of 9.7% is close to OMC obtained by the standard Proctor compaction (9.5% for Blend 2 and 9% for Blend 4). At such a high degree of saturation, the surface tension on particles caused by moisture is expected to reduce the segregation of particles. This aligns with the laboratory observations of the sand-rain technique compactions undertaken at a height of drop of 2m.

Installation of optic fibre (O/F) cables will be carried out after backfilling. Figure 10 shows the indicative location of O/F cables each having several sensing points with 500mm spacing. The cables will be installed using a CPT rig by attaching the cable to the CPT rod and pushing the rod to the required depth. All cables will be connected to a data acquisition unit located on the surface using which settlement data can be collected in each site visit.

4 CONCLUSION

Blends of recycled materials including recycled glass, recycled plastic and tyre derived aggregates were tested to investigate the feasibility of replacing these aggregates with the conventional backfill material in deep sewer trenches. Several laboratory tests including the sand rain technique were carried out to investigate the physical and mechanical properties of the four blends. Results showed that the dry density increased as moisture content and height of drop increased. Also, the dry density decreased as the tyre content increased. The MDD was achieved when a blend was prepared at a moisture content $\geq 15\%$ and after dropping the blends from a height of 2m. At high moisture contents, the MDD obtained through the sand-rain technique approached the corresponding values obtained from the Proctor compaction and achieved $> 80\%$ relative density.

Achieving greater relative density and exhibiting less compressibility response, Blends 2 and 4 were recommended for the construction of the trial sites.

To avoid the ingress of excess water into the natural ground due to moisture contents higher than the field capacity, backfilling of the sites with Blends 2 and 4 mixed at the MCs corresponding to their field capacity and height of drop of at least 2m is recommended for the construction of the trial sites.

Two possible downfalls with the proposed application recycled blends include potential creep settlement and environmental concerns, such as leachate seeping into the groundwater. These potential issues may limit the application of the proposed blends to trenches with low surcharges on the backfilled zone (such as non-trafficable areas) and areas away from groundwater table. The creep behaviour and leaching properties of the blends will be investigated as part of the future laboratory experiments of this project.

At the time of preparing this paper, the construction arrangements are being made for the next stage of the project.

5 ACKNOWLEDGEMENTS

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SESSION 4
TECHNOLOGY BASED
SOLUTIONS

Innovative Geotechnical Design, Smart Construction and Optic Fibre Sensing

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ABSTRACT

The future of built infrastructure including geotechnical structures relies on smarter use of information to make better engineering decisions leading to a reduction in risk, costs and carbon footprint and enhanced resilience and integrity. The rich information obtained from emerging sensor technologies can act as a catalyst for innovative geotechnical design, smarter construction planning and more cost-effective and energy efficient construction processes. Design of geotechnical structures requires consideration of risks associated with variable geology, which can be managed by having high spatial resolution data to understand the effects of varying soil-structure interaction. At present, there is a mismatch between the data needed and what current sensing systems can collect, which makes data-driven decision hard to achieve in real-time. Fibre optic sensing (FOS) is capable of obtaining high spatial resolution data in geotechnical engineering projects, allowing real-time acquisition of large volumes of data that can be analysed and fed back to designers and constructors to improve construction performance. This paper presents a review of the role of data in driving smart, cost-effective and energy-efficient design and construction and in particular discusses how optic fibre sensing can facilitate this in geotechnical engineering projects. Three local case studies are presented to demonstrate the capability of fibre optic monitoring. The first case describes the use of FOS in developing a full picture of soil-structure interaction on driven mini-piles during pile load tests, enabling the development of new analytical solutions for this footing system. The second case presents monitoring of a pipe-jacking project in Melbourne where a Glass-fibre Reinforced Plastic pipe was instrumented with FOS to capture its behaviour during the drive. The measured longitudinal and circumferential strains led to better understanding of the effects of the process on the pipes, providing a foundation for design improvement. The third case demonstrates the capability of FOS in long-term monitoring of elastic and permanent deformations in pavements constructed with recovered waste tyres.

Keywords: geotechnical field instrumentation, fibre optic sensing, soil-structure interaction, data-driven design

1 INTRODUCTION

In recent years, sensing technology research has undergone a revolution, especially in the built infrastructure environment. Effective use of monitoring systems can provide essential information for innovative design and effective use of new materials (Soga and Schooling 2016). There are numerous conventional geotechnical monitoring systems adopted over the years to monitor geo-structures at discrete locations, such as vibrating wire strain gauges to measure one dimensional strain (Tedd et al. 1984), inclinometers and tiltmeters to measure relative deflection and bending moments (Clark & Richards, 2005) and automated total stations to measure 3D displacements (Beth et al. 2003). However, a major drawback of these monitoring tools is their discrete nature leaving large areas of structures un-monitored and as a result, relevant structural behaviours or defects cannot be captured (Lienhart et al. 2018). Further, as these instruments are influenced by various external factors such as humidity, temperature and depth (Mikkelsen 2003), a feasible, cost effective and robust monitoring system is needed to tackle the challenges faced by currently available monitoring systems.

Fibre optic sensing (FOS) has emerged as a strong, robust and accurate sensing technology (Soga and Schooling 2016) with the capacity to provide site-scale mechanical and thermal monitoring for applications which require continuous measurement, high spatial resolution of sensing, or span over long distances. Innovations and usages of fibre optic sensors in "structural health monitoring" of structures have been

increasing because of the advantages that fibre optic sensors provide (Huybrechts et al. 2016). In geotechnical structure monitoring, critical aspects such as fibre-soil interaction and fibre adhesion to the structures needs to be carefully considered since these factors can have a significant impact on the strain transfer efficiency from soil to fibre optic sensors and hence the structure (Zhang et al. 2020).

This study provides a summary of three unique local case studies conducted in Australia to demonstrate the capability of fibre optic monitoring in improving designs and facilitating the use of new material for geotechnical engineering.

2 SENSING TECHNIQUE

2.1 Fibre Bragg grating

Fibre Bragg grating (FBG) is a quasi-distributed sensor solution. The sensors are typically deployed along the length of an optical fibre. In this technique, the ultraviolet laser's interference pattern is used so that the periodic structure of FBG can be generated (Meltz et al. 1989). When an incident spectrum of light propagates through the grating, as shown in Figure 1 one of the wavelengths called Bragg wavelength turns back while the remaining spectrum is not affected. When external axial strain takes place along the fibre, the FBG will respond accordingly, resulting in a proportional shift in the reflected Bragg wavelength. This shift can be converted to an equivalent strain through a one-time instrument calibration. The biggest advantage of FBG technology is that multiple measurement points can be set along one optical fibre

to create an array of independent sensors, thereby enabling efficient multiplexing and distributed measurement. Also, these multiplexed FBG sensors can be applied to monitor other parameters, such as strain and temperature in real time (FBG Principle 2020).

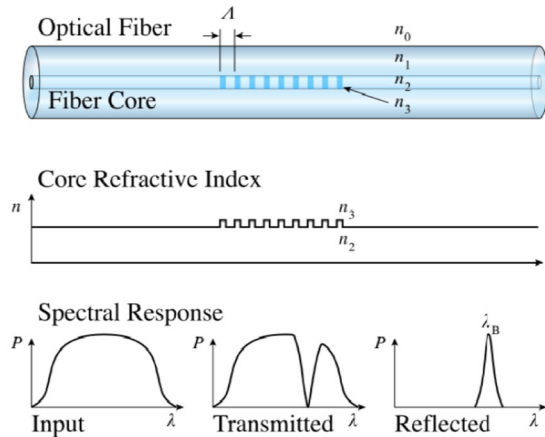


Figure 1. FBG Sensors Operating Principle (Rao 1998)

2.2 Installation methodology

A bare optical fibre is fragile and might not survive in harsh field conditions (Hong and Zhang 2016). Therefore, secure packaging techniques should be adopted. In the projects described in this study, the fibre optic cables were mounted onto the structures using a two-part epoxy adhesive. This adhesive requires 12 to 15 minutes curing time to achieve 20.7 to 27.6 MPa tensile strength. For the fibre packaged with adhesive, it is vital to consider the impact on strain transfer from the structure to embedded arrays (Zhang et al. 2020). In practice, a strain transfer efficiency factor should be applied, and this parameter can be obtained through laboratory or field calibration tests.

3 CASE STUDIES

Three local monitoring projects around Australia are presented. These projects utilised FBG sensors as the principal instrumentation technique to enhance our understanding of soil-structure interaction effects. The first case study describes the use of FOS in developing a full picture of soil-structure interaction on driven mini-piles during pile load testing, enabling the verification of analytical solutions for this footing system. The second case study presents some results from a pipe-jacked Glass-fibre Reinforced Plastic (GFRP) pipe monitored using FOS to assess its performance during the drive. The measured longitudinal and circumferential strains revealed the deformation pattern of GFRP pipe at each stage of the jacking process, which led to a better understanding of the process. The third case study demonstrates the capability of FOS in long-term monitoring of elastic and permanent deformations in pavements made with recovered waste tyres.

4 CASE STUDY 1: DRIVEN MINI-PILE

4.1 Project description

Field pile load tests were performed on mini-piles instrumented with fibre optic sensors to understand the degree of interaction with the surrounding soil. The test site is located in Fingal, Victoria, Australia. Three Cone Penetration Tests (CPT) were conducted within the test area. Figure 2 shows the results of the CPTs. Two CPTs were terminated at a depth of approximately 2.5 m, where a relatively dense layer of silty sand was encountered. The soil profile primarily consists of medium dense silty sand and the soil profile is consistent up to a depth of 2 m. No groundwater was encountered up to 10 m depth.

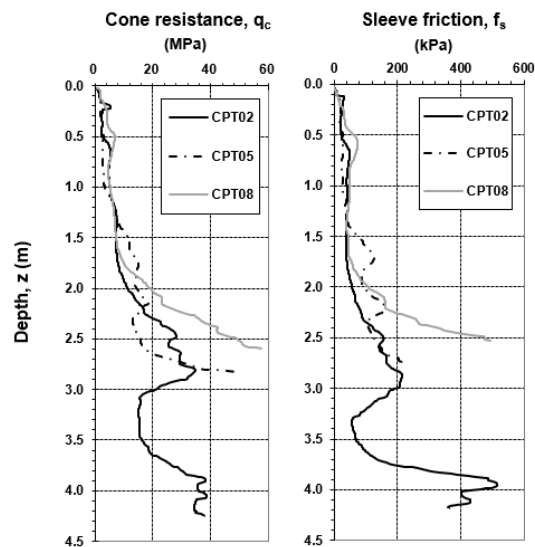


Figure 2. Results of CPTs performed at the test site (Tsang et al. 2019)

4.2 Monitoring motivation and purpose

Vertically loaded battered (raked) piles are used in various geotechnical applications, such as slope stabilisation, earth retention and bridge abutments. Most published studies describe the load response of battered mini-piles in model pile load tests (Tsukada et al. 2006, Kyung and Lee 2017). There are few studies reported that address fully instrumented field load tests to monitor the behaviour of battered mini piles. The aim of this study was to understand the soil-pile interaction of a battered driven mini-pile by monitoring the strain development along pile's shaft during field load tests.

4.3 Instrumentation

The test mini pile is a steel, open-ended pipe pile with an outer diameter of 42.4 mm and a wall thickness of 2.6 mm. Three grooves, namely G1, G2 and G3, 4 mm wide and 1.5 mm deep were machined along each test pile shaft (Figure 3a) to mount FBG sensors. A fibre string with a total of six FBG sensors was embedded in each groove. Locations of the six sensors are provided in Figure 3b.

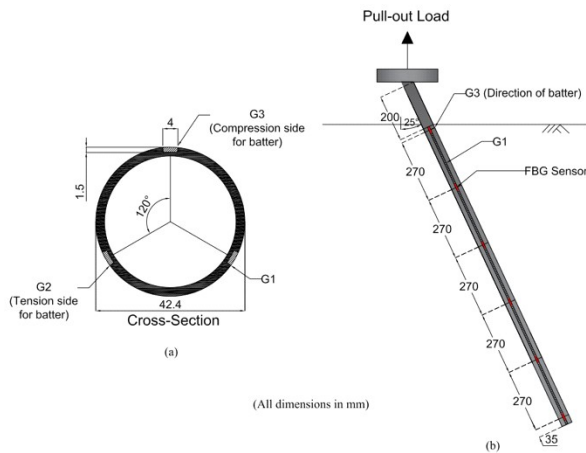


Figure 3. Dimension of grooves along test pile's shaft and layout of FBG sensors inside the pile: (a) Cross-section of pile and configuration of grooves; (b) layout of FBG sensors along battered mini pile length (Tsang et al. 2020a).

4.4 Results and analysis

Figure 4 shows measured strains in the fibres along the pile shaft. It can be seen that there exists a point of rotation when the battered pile was subjected to predominantly vertical load as indicated by the development of tensile strains measured along G3.

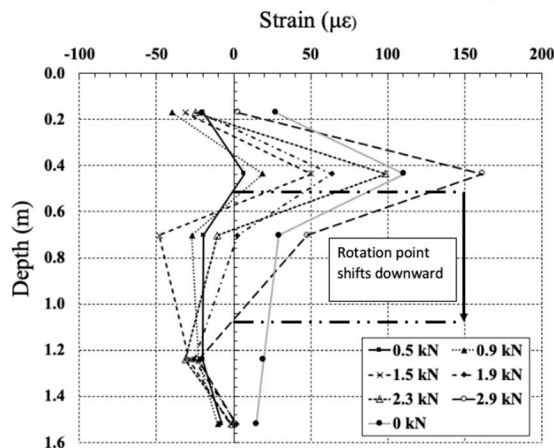


Figure 4. An example of measured strain curves along G3 during pull-out load stage (Tsang et al. 2019).

The measured point of rotation along the pile length (i.e. point where strain changed from positive to negative) shifted from 330 mm to 890 mm below ground surface with increasing load magnitude. Prasad and Chari (1999) has proposed Equation (1) to calculate the depth of point of rotation (a in metres) when a pile is subjected to ultimate lateral load.

$$a = \frac{[-(0.567L+2.7e)+(5.307L^2+7.29e^2+10.541eL)^{0.5}]}{2.1996} \quad (1)$$

where e (m) is eccentricity of loading; and L (m) is embedment depth of pile. The measured depth of rotation point was then compared with the calculated value. The calculated value was 970 mm which shows good agreement with the measured value with a difference of 80 mm, or 5%. These preliminary FBG measurements show the shift of rotation point as load level increases, which helps determining the pile soil interaction at service load rather than just ultimate

load. This project proved the feasibility of using FBG sensors in mini pile monitoring, which also highlights the importance of conducting field monitoring when developing analytical solutions.

5 CASE STUDY 2: PIPE-JACKING

5.1 Project description

A GFRP pipe section was instrumented with fibre optic sensors to assess its field performance during the pipe-jacking process. This project comprised installation of a new sewer main pipe, approximately 1160 m in length. The instrumented section of sewer was launched from shaft KAL-10, at an invert depth of approx. 12.4 m with groundwater level of 11.5 m below ground level. Figure 5 indicates the pipeline alignment for the 580 m section of drive from shaft KAL-10 to KAL-11. Data was collected over discrete periods, commencing 100 m from shaft KAL-10.

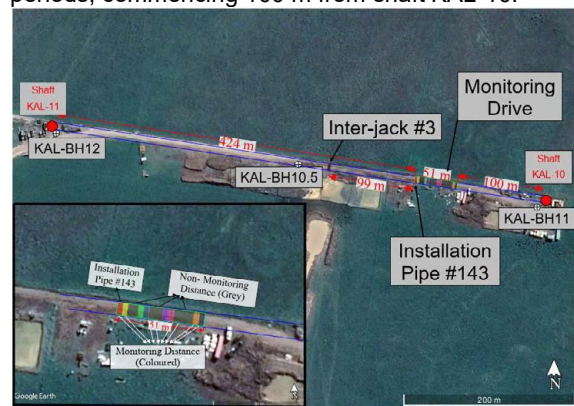


Figure 5. Pipe alignment and monitoring plan view (Tsang et al. 2020b)

5.2 Monitoring motivation and purpose

GFRP pipes have several benefits over traditional reinforced concrete pipes including greater tolerance to changes in direction and flexure, as well as higher compressive strength which reduces pipe thickness. However, previous investigations of this material were limited to controlled laboratory studies, and there is a lack of data to assess its field behaviour in construction (Robinson 2016). The aim of the study was to monitor and analyse the deformation and structural performance of a single GFRP pipe section during the jacking process taking into account the complexity of field conditions and the pipe-jacking process.

5.3 Instrumentation

In this project, twenty-four metres of field monitoring data was obtained from a 51 m length of drive. The instrumented GFRP pipe (number #143) was 3 m in length, 1.6 m inner diameter and 1.72 m outer diameter. The instrumentation involved 24 monitoring locations, comprising five FBGs at intervals along three circumferential cables,

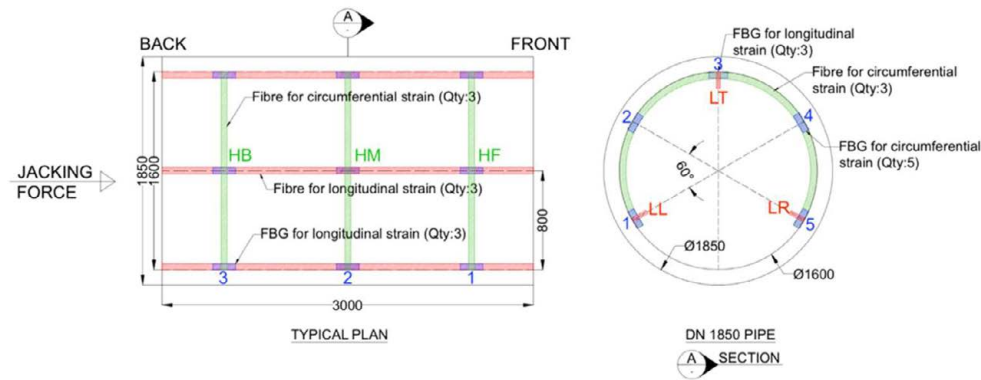


Figure 6. FBG sensor configuration (a) plan view and (b) cross-section view (dimensions in mm) (Jiang et al. 2019)

and three FBGs along three longitudinal cables attached to the intrados of the GFRP pipe as shown in Figure 6. For referencing purpose, the circumferential (or hoop strain) FBGs (with green hatch) were named Hoop Front (HF), Hoop Middle (HM), Hoop Back (HB) in the drive direction. In each circumferential array, the FBGs were numbered 1 to 5, clockwise from knee to knee, again viewing in the direction of the drive.

compared as shown in Figure 7. The strain at positions 1 and 2 are compared as an example of strain development during the jacking process. Significant patterns include first, HF1 experiences relatively higher compressive strain during the entire field monitoring period compared to HF2. Second, HM2 remains in compression which is the same as the behaviour of HF ring. Third, HM1 is the only FBG that experiences tensile strain, which is opposite to the behaviour observed at HF ring. Figure 8 shows the relative radial strains around the instrumented pipe at HF and HM arrays respectively.

5.4 Results and analysis

The hoop strains measured from the HF (front array) and HM (middle array) fibres are analysed and

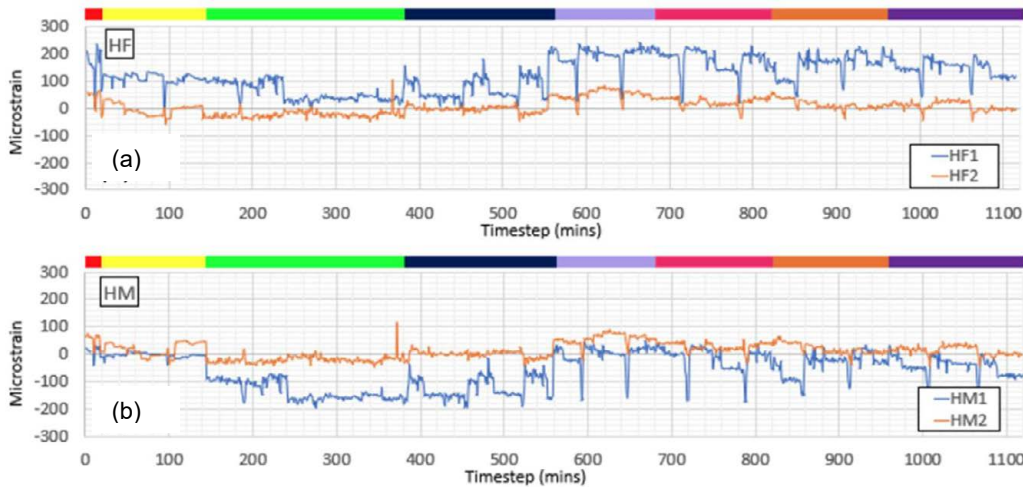


Figure 7. An example of measured strains around the circumferences of the pipe at: (a) HF (front ring), HF1 vs HF2 and (b) HM (middle ring), HM1 vs HM2 (Tsang et al. 2020b)

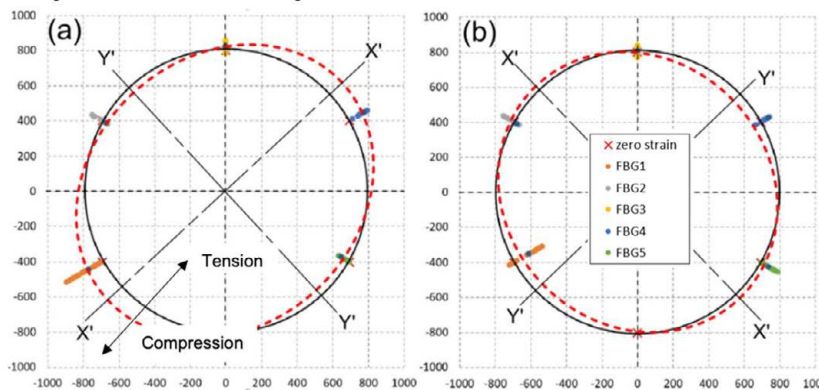


Figure 8. An example of radial strain development from measured hoop strain at chainage 143, 144, 147 and 148: (a) HF; and (b) HM (Tsang et al. 2020b)

By plotting the incremental hoops strains at the position of each FBG sensor, Figure 8 shows the development of hoop strains over the jacking period. A snapshot of the pipe distortion pattern at a specific time increment is represented by the dashed red line. In this case, Figure 8 a & b shows the distortion pattern 45 minutes after initial driving of pipe #144 (i.e. the pipe behind the instrumented pipe). From Figure 8, it can also be observed that there is an opposing structural behaviour between HF and HM arrays, which are only 1 m apart, indicating non-axisymmetric 3D distortion along the pipe. This is inferred to be due to the effects of variable ground-structure interactions such as soil friction, soil contact area, slurry fluid lubrication and jack loading pattern.

6 CASE STUDY 3: PERMEABLE PAVEMENT

6.1 Project description

A full-scale waste-tyre based permeable pavement trial site was constructed and monitored at a carpark located at St Marys Park, City of Mitcham, South Australia. Figure 9 shows the typical cross section of the waste tyre-based permeable pavement used in this project.

While three different mixtures with varying tyre-derived aggregate (TDA) content, i.e. 30%, 40%, and 50%, were used as the surface layer to study their mechanical performance, the underlying layers, i.e. screening, subgrade, and geosynthetics reinforcement, had the same mechanical properties and configuration throughout the monitored area.

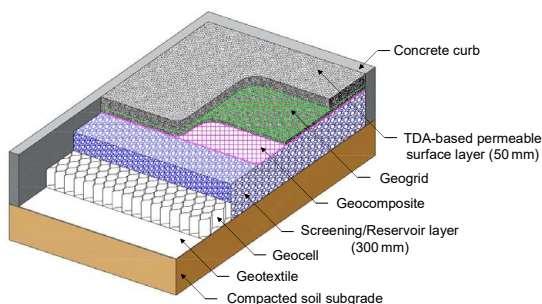


Figure 9. Typical cross-section of the pavement (Raeesi et al. 2020)

6.2 Monitoring motivation and purpose

In this application the objective was to observe how the “waste tyre permeable pavement”, developed at Melbourne School of Engineering, performed under real field conditions with traffic load and environmental loads both present. Because the exact location of a car tyre in a parking spot is hard to predict, multiple sensors with high spatial resolution were required to be able to capture any possible loading condition; hence optic fibre technology was used in place of hundreds of strain gauge sensors.

6.3 Instrumentation

In this project, two arrays of optic fibre were installed at each of the three instrumented carpark bays: one, with seven FBGs, located at the depth of two thirds of

the thickness of the surface layer, and the other array, with five FBGs, located on the top of the screening layer as shown in Figure 10.

In both cases, the optic fibre was mounted on a thin metal strip. To assure that the recorded shift in the wavelength was transformed to strain by the correct ratio, a strain gauge was installed next to one of FBGs on the strip.

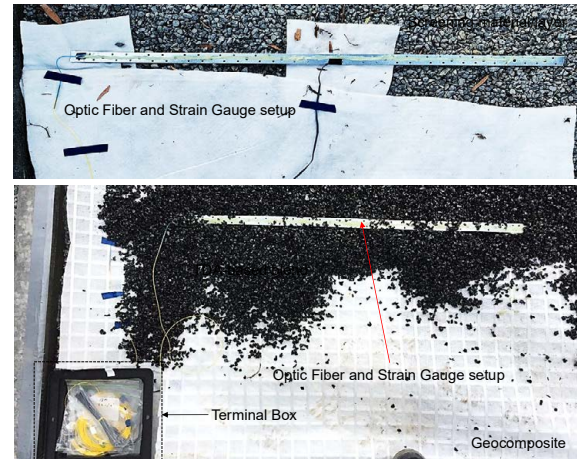


Figure 10. Installation of instruments for monitoring

6.4 Results and analysis

Figure 11 shows an example of the measured strain profiles at different depths. It can be observed that the peak strain reduces as the depth increases, which is in agreement with the stress-strain distribution in layered semi-finite systems. The measurements indicate that a decrease in the TDA content results in decrease in strains developed in both surface and screening layers.

The ratio of the peak strain measured at the interface to that measured in the surface layer is 29%, 18% and 11% for sections with 30%, 40%, and 50% TDA, respectively. This shows that ratio of the surface-to-bottom layer moduli should be decreasing as the TDA content increases. Considering the fact that the only changing factor between the monitored sections is the TDA content in the surface layer, it can be concluded that the modulus of the surface layer significantly increases, as the TDA content decreases from 50% to 30%. This was also verified by the results of other tests done at the carpark.

7 CONCLUSIONS

Three field performance monitoring projects of geotechnical structures using FOS technology are presented and discussed in this study. It is shown that FOS allows collection of data which cannot be easily obtained by conventional sensing techniques, especially for projects in harsh environments. This study provides some examples of data collected with fibre optic strain monitoring system in each project. The finding in each field monitoring scheme is summarised below:

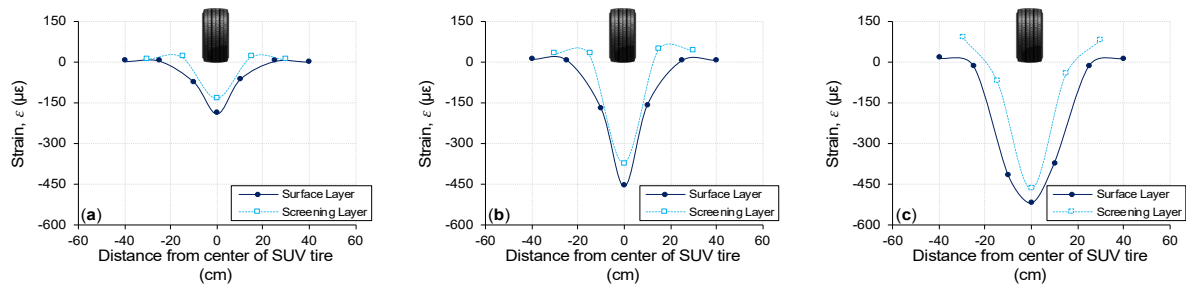


Figure 11. Strains measured at sections with a) 30%, b) 40%, & c) 50% TDA in the surface (Raeesi et al. 2020)

1. Mini driven pile: the soil-pile interaction of a battered mini pile was investigated, focusing on the strain development during field pile load testing. The measured strain curve matches the theoretical point of rotation which provides essential information for analytical solution development for service conditions.
2. Pipe-jacking: A GFRP pipe section was instrumented to study its field behaviour during pipe-jacking process. The data shows that a non-uniform diametric deformation occurs during jacking. The compression and tensile strains on the pipe intrados show that the pipe is subjected to inclined ovalisations which vary with longitudinal position along the pipe. This is a factor which is not considered in pipe-jacking design. The collected data allows assessment of the field performance of this relatively new pipe material, as well as providing data required for improved construction control.
3. Permeable pavement: A waste-tyre based permeable pavement was instrumented to assess the strain generated in layers with various tyre content. The FOS system was proved to work well in the aggressive pavement environment where humidity is typically high. Results indicate that the strain generated decreased with decreasing TDA content.

8 ACKNOWLEDGEMENTS

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An Innovative Approach for the Installation of Vibrating Wire Piezometers

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ABSTRACT

Vibrating wire piezometers, described hence forth as VWP, are high precision geotechnical instruments installed into the ground to electronically monitor pore water pressures over extended periods of time. These instruments have use in a variety of geotechnical engineering settings such as: slope stability; basement excavation; consolidation under embankments and reclamations; and the performance of hydraulic structures e.g. dams. The piezometric data are useful for the design of these structures and for monitoring the performance during construction and in service. VWPs are traditionally grouted into drilled bore holes in a process that produces waste spoil and requires the use of additional materials for back filling. This paper discusses and reviews two case studies of an innovative, low impact and high-quality approach for the preparation and direct push installation of the VWPs. This installation approach eliminates the production of spoil, is fast and therefore low cost, completed with low impact plant and most importantly has a very high success rate using a locally manufactured push in adaptor and off the shelf VWPs.

Keywords: vibrating wire piezometer, ground water, in situ testing

1 INTRODUCTION

Vibrating Wire Piezometers are utilised in a variety of geotechnical engineering settings to provide information during the design, construction and in service of pore water pressures, at the sensor location.

VWPs have traditionally been installed directly into boreholes using a filter sand intake zone and bentonite seal or into a fully grouted borehole. The sand filter and bentonite seal technique in particular is prone to difficulties in the placement of backfill materials leading to installations ending up less than satisfactory (Mikkelson 2002). In situations where the VWPs are fully grouted into position, it is important that the grout properties are matched to the permeability and strength of the surrounding formation (Yungwirth 2013).

More recently the 'nude' VWP installation technique has been adopted on occasions. After placement of the VWP into a cased bore hole, this method relies upon the complete collapse of the borehole (Mazur 2015). This approach is contingent on the collapse of the bore hole around the VWP and not suitable in many conditions.

Another approach for installing VWP is to directly push them into the ground using the crowd forces of the drill rig. Whilst this approach has been available for some time, limitation with installation forces and the need to rotate rods to decouple from the VWP increases the risk of damage during installation. Several manufacturers recommend the VWPs are pushed a short distance into soft cohesive soils only, once again limiting the potential application.

This paper discusses an innovative technique using an in-situ testing rig fitted with a high capacity hydraulic pusher and small diameter Cone Penetration Testing (CPT) rods to push the VWP into the ground. A purpose built, high strength adaptor is

required to enable installation using this approach and is discussed further in this paper.

This direct push technique eliminates the production of spoil, is fast and therefore low cost, completed with low impact plant, does not involve complicated grouting and most importantly has a very high success rate using a locally manufactured push in adaptor and off the shelf VWP.

2 MAIN TEXT

2.1 Basic Principle of a VWP

A VWP utilises a sensitive stainless-steel diaphragm to which a vibrating wire element is connected. The diaphragm is protected by a filter which is used to keep solid particles away from the diaphragm. There is a small gap between the filter and the diaphragm which requires saturation to provide rapid response to changes in pore pressures (refer Figure 1). During use, changes in pressure applied on the diaphragm by water cause it to deflect. This deflection is measured as a change in tension and frequency of vibration of the vibrating wire element located behind the diaphragm. The square of the vibration frequency is directly proportional to the pressure applied to the diaphragm (Geokon 2018).

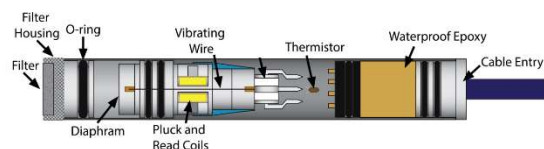


Figure 1. Typical Vibrating Wire Piezometer

The vibration signal is transmitted through a cable to either a portable handheld readout unit or to a longer-term data logging system.

2.2 Sacrificial Direct Push Adaptor

The speciality sacrificial push in adaptor was designed primarily to: (a) permit the use of almost any off the shelf 19mm diameter VWP; (b) protect the VWP and its filter from excessive bending and compressive stresses that can be generated during installation, noting that some direct push machines have the capability to mobilise 20t of pushing force and; (c) to positively seal the device in the ground when the CPT rods are extracted. This final point is important to note as this device does not require grouting into position or rely upon the collapse of the ground above to seal the VWP in the ground, reducing the possibility of seal bypass and the VWP reading pore pressures from outside of the target zone.

The dimensions of the cone tip, inner tube and outer tube have been carefully selected to minimise smear, protect the VWP slot, seal the device in the ground and aid simple installation. The adaptor is shown schematically in Figure 2.

These dimensions allow for a compression only connection to the CPT rods, resulting in an adaptor that stays in place upon extraction of the CPT rods from the ground, and the elimination of rod rotation to disconnect from the adaptor which can damage cables.

Two 1mm wide by 20mm long slots are positioned at the bottom of the adaptor to allow water to migrate towards the VWP filter.



Figure 2. Direct Push Adaptor with VWP mounted internally

The adaptors are manufactured locally from 316 stainless steel. This reduces lead times when compared to other adaptors coming from overseas.

Despite being durable, there is no intention or requirement for the adaptor to have long term durability.

2.3 Preparation of VWP in Adaptor

The filter on the VWP is carefully removed from the piezometer body by twisting and pulling. The filter and the VWP body are then submerged in water for 30 minutes to allow for temperature stabilisation and saturation before being carefully reassembled for an

'Initial Zero Reading' check of the VWP. For lower capacity VWPs (less than 700kPa), the pressure readings are collected during reassembly to ensure that the diaphragm is not overloaded. This value is compared to the 'Factory zero reading' detailed on the calibration certificate to ensure the VWP is in appropriate working condition.

The adaptor is then partially filled with dry coarse sand from the base to above the water entry slots. This small amount of sand is utilised to further protect the filter. Once this Initial Zero Reading check is completed the off the shelf 19mm VWP is placed inside the adaptor so that the filter of the VWP is bearing on the sand at the tip end. Some additional sand is added around the tip of the VWP to ensure there is embedment in the sand. As the VWP will become de-saturated in this process an additional re-saturation is conducted on the conclusion of the preparation.

Melted microcrystalline wax is poured down the inner tube to a level approximately 50mm above VWP, securing the VWP in position within the adaptor.

Crushed bentonite is then placed within the inner tube to fill the remaining annulus to approximately 100mm from the top end of inner tube. Some tapping of the outer tube aids the bentonite to settle inside the tube.

While holding the VWP cable centrally in the inner tube, a final seal of wax is poured inside the tube to completely fill and seal the inner tube.

This preparation process is utilised to ensure moisture from above does not impact on pore water pressures measured at the tip.

The above preparation process can be conducted on site within the in-situ testing rig or prior to arriving on site at a workshop, site shed or similar.

Prior to site installation the tip end is submerged in water for a minimum of 30 minutes to re-saturate the sand, filter and annulus between the filter and the diaphragm. During this process water and air are compressed into the tip end, after a period of time the air will dissolve into the water, filling the filter and the space above it entirely with water (Model 4500 Piezometer Manual). A final "on-site" Initial Zero Reading using the hand held readout unit is conducted. This value can be compared to the initial reading to demonstrate the VWP is in working condition after placement within the adaptor.

2.4 Plant Utilised for the Direct Push Installation

Purpose built direct push rigs, typically used to conduct various forms of in situ testing can be utilised to install VWP using this approach (refer Figure 3).

The common features of these machines include high mobility, low environmental impact, low track or tyre pressures, short setup durations, high hydraulic pushing forces and typically high reaction forces that can be mobilised when required.

These machines are generally readily available at all major centres throughout Australia and can be mobilised to remote locations.



Figure 3. Typical in-situ testing and sampling rig

2.5 Direct Push Installation

The in-situ testing rig used to install the VWP is set up on location. The pushing frame is then adjusted so that the force generated is perpendicular to the ground. This adjustment is made to reduce bending forces on rods that can be generated during the pushing.

In order to mitigate the risk of generating pore water pressures greater than the capacity of the VWP during the installation an initial 'dummy' push is conducted.

In this process CPT rods with either a CPT cone or dummy tip are pushed into the ground using the pushing frame. When using a CPT cone details of the soil profile and hole verticality can be measured. The target of this initial dummy push is 1.0m above the nominated VWP installation depth. The rods with cone or dummy tip are removed from the ground leaving an annulus for the subsequent VWP installation. No spoil is generated during this process.

The VWP cable is threaded through the CPT rods and the VWP adaptor secured to the lead rod. A pre installation check of the VWP is conducted using the handheld Vibrating Wire Readout Monitor and recorded on an Inspection and Test Plan Check Sheet. This reading becomes the initial zero reading for the VWP.

The VWP, secured inside the VWP adaptor, is then pushed into the ground to the required depth. Again, no spoil is generated during this process and because the adaptor is sealed in the ground no cement-bentonite grout seal is required.

It should be noted that where there are concerns regarding confined aquifer conditions within the soil profile, a small quantity of bentonite cement slurry can be poured down the centre of the rods during the extraction process (much like a traditional tremie pour) to fill the annulus left by the CPT rods.

The above process to install a VWP, once prepared, occurs in a time period not much longer than a typical CPT sounding.

On conclusion of the installation any excess cables are coiled up and secured above ground level. The in-situ testing equipment leaves the installation location, without generating any spoil, using limited water and often without creating any discernible disturbance of the surroundings.

2.6 Site Example #1

A site investigation was conducted for a new facility located near the banks of the Markham River in the Morobe Province approximately east of Lae, Papua New Guinea (PNG). The area has annual average rainfall of 4.3m and has high levels of seismicity.

The remote location meant innovative approaches for conducting the investigation were required. A portable in-situ testing and sampling pushing system was mobilised to site, with reaction force provided by a locally sourced excavator (refer Figure 4). The excavator was also utilised to clear vegetation at testing locations, and handle plant and equipment on the site.



Figure 4. Portable in-situ testing rig supported by excavator

Initial CPTu probing was conducted across the site to profile soil conditions and aid decision making for further testing and VWP installation. The finalised VWP depths were chosen by the Geotechnical Consultant based on a combination of project requirements and soil conditions encountered on the site. Typical CPT data from the site are presented in Figure 5.

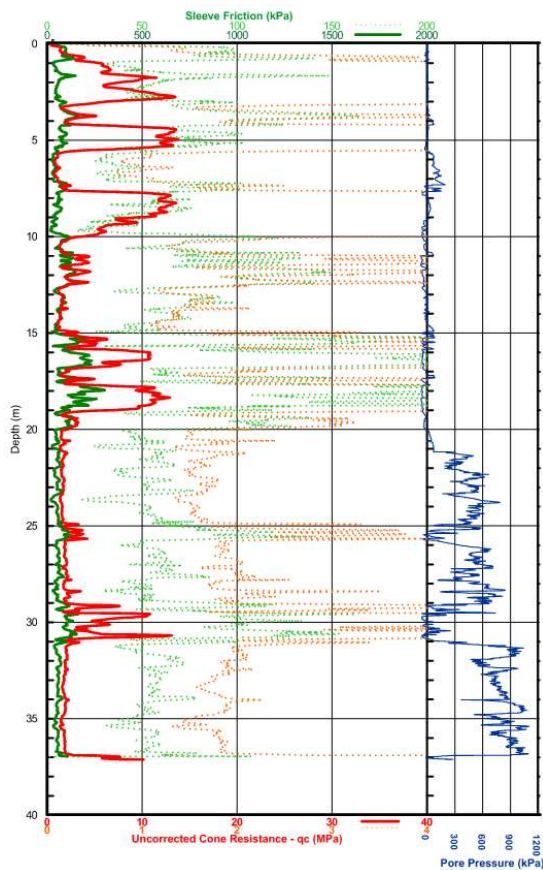


Figure 5. Typical CPTu sounding from project site (Site Example #1)

VWPs were procured and prepared within the VWP adaptor within the controlled environment of a workshop in Australia prior to transport into PNG. The prepared VWPs were transported in padded Pelican cases to mitigate the risk of damage.

The on-site installation was conducted in accordance with the procedure detailed above using the portable pushing frame. The VWPs were pushed to a depth of 20m below the existing ground level.

On completion the VWPs were connected to individual data loggers secured within an IP66 enclosure. The logger was programmed to collect data at the desired frequency. The data collected by the VWP over time was in line with expectations. An example of the data collected is presented in Figure 6.

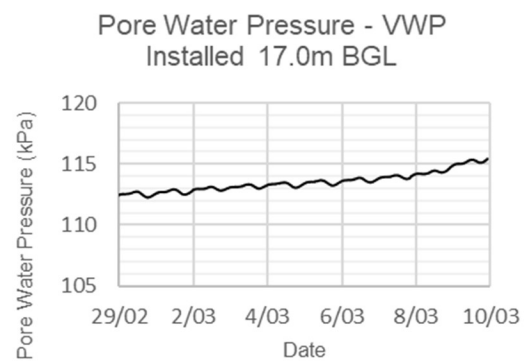


Figure 6. Pore Water Pressure data

2.7 Site Example #2

A site investigation was required on a tailings storage facility in remote central Australia where access to water was restricted at the time of the investigation. The purpose of this investigation was to confirm design parameters and to allow monitoring of conditions during the construction of an embankment raise.

Initially an extremely lightweight, extremely low track bearing pressure CPT rig conducted an investigation of the tailings crust using shallow CPT profiling. This initial testing indicated that the strength of the tailings crust was suitable to safely support a larger and heavier in situ testing rig capable of mobilising greater pushing forces.

The investigation consisted of CPTu testing, Dilatometer testing (DMT) and direct push Vane Shear Testing of the tailings. This testing was conducted from the low strength tailings surface with the in-situ testing rig tethered to support vehicles on the existing embankment. Typical CPT data from the site are presented in Figure 7.

The tethering of the in-situ testing rig to an on land 'fixed' point enables the quick and safe recovery should localised failures of the tailings crust occur.

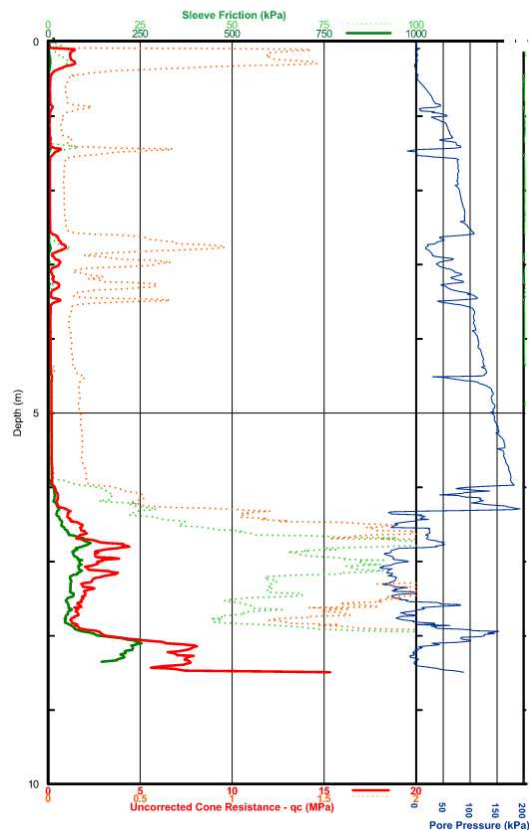


Figure 6. Typical CPTu sounding from project site (Site Example #2)

On completion of the site investigation component of the works, VWP's were installed on the upstream side of the embankment. The purpose of the VWP's was to monitor changes in pore water pressure during construction loading.

For this particular project there was limited site facilities due to the remote location. As such the VWP's were prepared on site the day before installation. Re-saturation of the VWP's was conducted in a bucket of water.

The in-situ testing rig again traversed across the tailings to the desired VWP installation location and successfully installed the VWP's to a depth of 5m below the existing surface level.

On completion, excess cable was coiled locally in a safe position for connection to a logging system by others.

Traditional investigation and VWP installation approaches would have required the mobilisation of earth moving equipment to construct causeways into the tailings to enable access for drilling plant, equipment and personnel. The drilling operation would have generated spoil which is then brought to the surface creating potential environmental disposal issues.

3 CONCLUSIONS

The direct push method with the utilisation of a purpose built adaptor has been utilised on numerous projects. It provides the project team confidence that the VWP is: installed at the correct depth without over stressing the VWP, is housed within a filter sand, is not impacted by smearing during installation and sealed from unexpected effects of ground water from above or below and therefore will provide reliable data. It reduces the requirements for complicated and problematic post grouting activities that may damage or impact on data reliability.

Furthermore, this method of VWP installation utilises low impact and readily available plant and equipment. These machines typically have low impacts on the work site, due to large low-pressure tyres or rubber tracks and therefore generally require limited additional access to be constructed. Once prepared the installation process is fast and efficient, minimising the time on site for machinery.

This installation approach does not generate any spoil, eliminating the risk of exposure to contaminated soils and issues associated with Acid Sulfate Soils. Due to being pushed directly into the ground, the use of a bentonite-cement grout slurry to secure the VWP in location are not required reducing material consumption.

4 ACKNOWLEDGEMENTS

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Sustainable Engineering Solution for Slope Stability with Anchor Reinforced Vegetation System (ARVS)

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ABSTRACT

Sustainability and resiliency are becoming more important in project design, with emphasis being placed on the environmental impact. Slope stability solutions should be designed to provide a low environmental impact to achieve long-term performance and overall project success. During design, it is important to consider factors such as durability, economics and environmental impacts. The Anchor Reinforced Vegetation System (ARVS) is recognised as a sustainable armouring and slope stability solution proving both surficial stability and erosion control at the same time. An ARVS is a component system consisting of a High Performance Turf Reinforcement Mat (HPTRM) to provide erosion protection and surficial strength, coupled with Percussion Driven Earth Anchors (PDEAs) for resistance to shallow-plane instability. The system is designed to optimise rapid vegetation growth and keep soil in place, thereby resisting mobilisation of soil masses associated with sliding failures of slopes. Key physical and material properties of the component system include optimal ultraviolet resistance, flexibility, and tensile strength of the HPTRM, along with its ability to promote vegetation establishment through increased soil and moisture retention. PDEAs can be selected in various lengths and strengths and are composed of corrosion-resistant material to ensure longevity while maintaining ease of installation. Design methodology of the ARVS for reinforcement against relatively shallow translational sliding failures consists of an infinite slope method solution adapted for the inclusion of PDEAs. Procedures for utilising the ARVS for relatively deep-seated rotational sliding failures include the modelling of stability using conventional limit equilibrium methods. Components of the ARVS are integrated into the model using slope stability modelling software. Results include the potential for an engineered ARVS solution for specific cases of reinforcement for slope stability. The sustainability of the ARVS solution including environmental parameters such as carbon footprint, economical and engineering aspects is analysed and compared with traditional solutions. An example of the practical application of the design methodology is demonstrated.

Keywords: Slope stability, Reinforced vegetation, Sustainable, ARVS

1 INTRODUCTION

Slope movements occur in a wide range of material types (including rock, debris, and earth) and mobilisation modes (falls, topples, slides, spreads, and flows) (Varnes, 1978). Time and costs associated with the maintenance, repair, and reconstruction of failing slopes are financial burdens. Conventionally acceptable methods of slope repair such as substantial excavation and replacement often experience degradation and undesirable environmental impacts. Alternative sustainable methods for stabilisation of failing slopes are thus often demanded which can include passive, in situ remediation techniques that incorporate vegetation as a component of the stabilisation strategy. An Anchor Reinforced Vegetation System (ARVS) is such a method explored herein, consisting of a High Performance Turf Reinforcement Mat (HPTRM) capable of promoting robust vegetative growth, secured permanently by an engineered arrangement of Percussion Driven Earth Anchors (PDEAs) to provide slope surface stability (normally up to 3m deep). Of course, this system is not designed to solve global stability issues.

The design of an ARVS consists of identifying the mechanism of failure (or potential failure) of an earthen slope composed of geometric, soil, and hydrologic properties specific to a localised area. Evaluation of the failure (or potential failure) surface

location is critical to the reinforcement design of the system. Once the failure mechanism and failure surface are understood, deterministic computational evaluation methods can be used to predict slope instability and provide mitigation design alternatives. Facilitating the establishment of vegetation within the ARVS provides additional elements of strength to alleviate soil mass mobilisation over time.

This paper presents details of the ARVS, design procedure and installation for slope stability. The paper will focus primarily on failures associated with sliding soils present within the uppermost layers of a slope including regression/erosion/sloughing type failures on public infrastructure projects such as road and rail cuttings or embankments.

Almost all sliding of soil masses are because of the combination of increased shear stress acting on the soil, along with reduced shear strength of the soil itself. Identifying the sole reason of a sliding slope failure is nearly impossible, as multiple factors can cause the increased stress and reduced soil strength which leads to a significant soil mobilisation. As a result, it is reasonable to assume that eliminating all identified causes of increased stress and reduced strength is not necessary for stabilisation. Instead, a strategic implementation of reinforcement associated with improving one or two key failure mechanisms can be an effective mechanism (Sowers and Sowers, 1961).

While erosion is technically not a mode of slope failure, it can often be considered as a root cause and can end up with similar undesirable consequences. Rainfall and surface runoff can detach and transport soil particles, resulting in rilling and gullies forming along a slope face. These indentations can eventually alter (i.e. steepen) the slope geometry and the ability of the soil to support itself, leading to the propagation of a surface failure. Furthermore, active erosion on a slope will greatly limit the establishment and growth of vegetation, resulting in reduced resistance to shear stress, as vegetation affects the cover factor in allowable shear stress calculations (Coppin and Richards, 1990; Kilgore and Cotton, 2005). While prevention or control of erosion does not provide an engineered reinforcement solution for slope stability, it may greatly reduce the cost and impact of proposed methods to mitigate slope failure.

2 MODES OF FAILURE AND REINFORCEMENT

Sliding failures can generally be classified into one of two modes, translational failure or rotational failure. Translational failure is a sliding mass along a plane of failure parallel to the surface, of minimal depth (e.g., surficial slope instability) while rotational failure is deeper-seated sloughing of a larger soil mass, usually in a circular-shaped failure plane (Varnes, 1978). It is important to be able to differentiate between the two modes prior to selecting a representative model for analysis and design.

Translational failure is typically identified as a scenario where the length of the failure surface is at least 10 times greater than its depth (Miller, 2012). Typically, the plane of failure is underlain by a stronger soil or bedrock which makes the soil movement approximately parallel to the original slope surface. This representation can be modelled as an infinite slope, where conditions are idealised and there are no defined upper and lower boundaries within the failure plane. Simplifying the scenario into this model allows the designer to introduce a reinforcement system that bolsters the resisting forces and stabilises the slope against translational sliding (Gray and Leiser, 1982).

Rotational failure is normally when the length of the failure surface is typically 3 to 6 times greater than its depth (Miller, 2012).

Reinforcement against both modes of slope failure consists of reducing shear stresses as well as increasing the shear strength of the soil mass. The ARVS can be used as either a passive or active reinforcement system which makes this solution a unique solution. Resistance to instability can be provided passively through full development of PDEA capacity after slope movement begins, or PDEAs can be post-tensioned and locked at a specified design load to function as an active reinforcement system.

PDEAs are not only effective in resisting the shallow shear forces associated with both modes of failure,

but they also force the potential rotational failure arcs deeper into the ground, thus producing a curved failure path with greater normal stress along the lower half. This effectively increases the shear resistance of the soil mass due to higher normal stress, thus drawing on more of the friction component of the soil (as the cohesion component is independent of the normal stress).

3 ANCHOR REINFORCED VEGETATION SYSTEM (ARVS)

The ARVS is a multi-component system that includes a High Performance Turf Reinforcement Mat (HPTRM) installed on the slope surface, permanently secured using Percussion Driven Earth Anchors (PDEAs) in a designed pattern, driven to a designated distance below the surface of the slope (Figure 1). Each component of the ARVS has a unique purpose enabling the whole system to serve as an effective solution to provide slope stability against both translational and rotational sliding failures. The purpose of the ARVS as a slope reinforcement solution includes utilising the PDEAs as a system of active or passive reinforcement to provide resistance to driving forces, incorporating the HPTRM as a high-tensile strength geosynthetic (typically more than 40kN/m in both directions) with very high UV stabilisation for distributing load amongst the PDEAs and providing further resistance to movements, and providing a permanent layer for the establishment and reinforcement of vegetation through the three-dimensional matrix of the HPTRM, which will control surface erosion and assist in resisting shear forces.

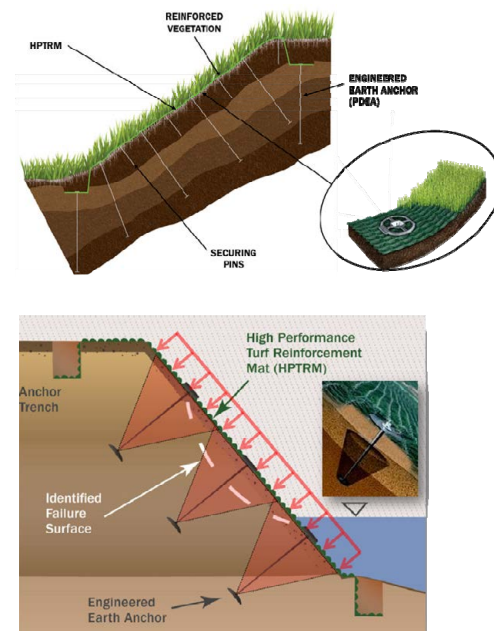


Figure 1. Anchored reinforced vegetation system (ARVS)

In addition, the ARVS serves to isolate the underlying soil from erosive forces by providing continuous coverage of the slope face. As the HPTRM is tensioned across the surface of the slope,

it has a compressive influence on the enclosed soil mass, thereby increasing stability against mobilisation. The tensioned geosynthetic also engages normal stress at the base of the failure surface, which increases stability of the soil mass. The PDEAs impart discrete loads at the slope face, and by penetrating through the failure surface provides friction, adhesion, bearing, bending resistance, and torsional resistance. The open weave of the HPTRM provides relief of pore water pressure build-up within the soil mass. Over time, the compressive HPTRM causes soil densification, thereby increasing internal friction angle and cohesion, providing greater stability. Finally, the HPTRM promotes vegetative growth (where applicable) and thereby provides extra benefits to slope stability (Koerner 1990; Hryciw and Haji-Ahmad, 1992).

Use of a synthetic HPTRM composed of a homogeneous, lofty, three-dimensional matrix of woven polypropylene fibres promotes the ability of the system to function as designed. A homogeneous dense woven HPTRM with the highest available tensile strength (minimum 40kN/m) should be used. Densifying the woven matrix promotes greater resiliency without sacrificing porosity of the geosynthetic to allow for relief of pore pressure from saturated soils within a slope mass (Figure 2) (Ghiassin et al., 1997). Hydraulic resistance to velocity and shear stresses (generated by flowing water), an optimal relationship between the overall thickness of the material and its flexibility, high strength and resiliency, and a high-degree of UV resistance (at least 90% retained tensile strength at an accelerated exposure simulation of 10,000 hours) are important parameters for the long-term performance and survivability of the ARVS.



Figure 2. Homogeneous dense woven HPTRM in an ARVS

PDEAs as part of an ARVS can come in a variety of sizes, lengths, and strengths, but typically consist of three major components. The first is a solid anchor head, typically manufactured as die-cast aluminium, and ranging in size and weight. The anchor head is connected freely to a tendon, which consists of either flexible wire rope or a rigid steel threaded bar. The length of the tendon is specified in the design, but typically requires embedment at least 600mm beyond the plane of failure. At the opposite end of the tendon is a bearing plate or cap, ranging in size from a 75mm diameter zinc-plated aluminium disc, to a 100mm x 150mm steel spreader plate (Figure 3).

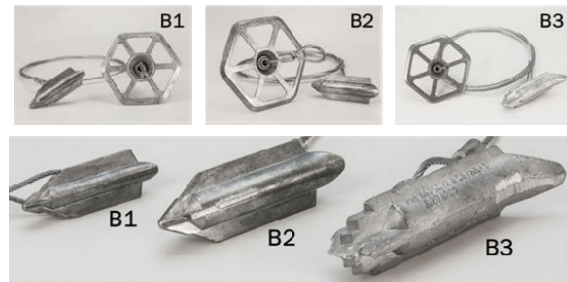


Figure 3. Percussion Driven Earth Anchors (PDEAs)

Understanding the mechanics behind installing the PDEA component will aid a quality installation of the whole system. The required tools are a percussion hammer, a ground rod driver, drive steel compatible with the PDEA, a setting tool, and wire cutters. The complete installation procedure and details should be followed which is out of the scope of this paper.

Strength of the PDEA is quantified by a value known as the “pull-out” resistance of the anchor, which is generally defined as the tension load causing one or more of the components to yield, or to physically remove the entire PDEA from the ground – whichever occurs first. Typically, the pull-out strength of the anchor is a function of soil characteristics, as removal of the PDEA from the ground normally occurs before components yield. Generally speaking, the pull-out strength of an anchor varies from 100-250kg for the smallest anchor (depending on soil characteristics), to approximately 2000kg for the largest (feasibly incorporated) anchor. In most cases, stability analysis and reinforcement design would be completed based on available geotechnical data, and subsequent tension loading and measurement would be performed on in situ soils of the slope to validate the design.

Establishment of quality vegetation can provide direct and substantial effects on the long-term stability of the ARVS-reinforced slope. Vegetation is critical to alleviating surface soil erosion, which can be a contributing factor to increased shear stress and decreased shear strength of a slope's soil. Vegetation serves to impede erosion through various means. Interception of rainfall is provided by the shoots, or foliage of a plant, absorbing its volume and dissipating its energy imparted to the soil. Restraint is provided by the roots of the plant, providing a system to physically bind and confine particles of soil. Retardation of runoff is provided by the shoots (foliage) of the plant by providing an increase of roughness to the slope surface, thereby slowing the velocity of erosive water flow. Infiltration of runoff is assisted by plant systems, which provide physical and chemical attributes that maintain healthy soil capable of infiltrating more water. Finally, transpiration of near-surface groundwater is achieved through plants that absorb it, providing delays to saturation of the slope's soils (Gray and Leiser, 1982). Selection of the appropriate geosynthetic material as part of the ARVS is critical to the performance of the system. Utilisation of an HPTRM with specific material and performance properties will increase the effectiveness of the ARVS with respect to erosion control, vegetation

establishment, tensioned load distribution and coverage, and functional longevity (Figure 4).



Figure 4. ARVS after installation (left) and after vegetation (right)

Although the erosion control and shallow plane mass-movement mitigation benefits of vegetation should always be incorporated where feasible, the HPTRM and PDEA array system in an unvegetated condition can still be an effective reinforcement solution in many cases (Figure 5). The UV resistance of the HPTRM is very important in these conditions. A minimum of 90% UV resistance after 10,000 hours accelerated test is suggested for the HPTRM. In cases that soil type and chemicals may affect the durability of the PDEA, this needs to be considered in the design and PDEAs made of special material may need to be used. As a substitute for the erosion control benefits of vegetation, the use of a lightweight, non-woven geotextile below the HPTRM can provide a barrier to the migration of fine aggregate particles, while still providing the open area to act as a porous medium to facilitate evaporation and drainage.



Figure 5. HPTRM and PDEA for an unvegetated condition

4 ARVS SOLUTION TO SHALLOW/SURFICIAL FAILURE (THE INFINITE SLOPE MODEL)

Where the failure mode is expected to be a relatively shallow, surficial failure, a basic infinite slope analysis is often appropriate. Available resistance to slope failure is dependent on the friction angle of the soil and the normal force N , which acts perpendicularly to the shear surface. However, additional frictional resistance can be realised by incorporating the anchor force to the equation, as illustrated in Figure 6, for non-saturated, cohesionless soils:

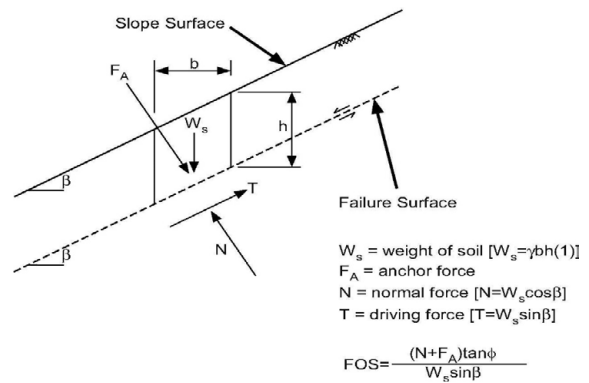


Figure 6. Infinite slope model calculation (Durham et al., 2012)

Where γ is soil unit weight, ϕ is soil angle of internal friction, c is soil cohesion, β is slope angle, h is vertical height of soil mantle, and b is width of the analysed element. The factor of safety (FS) against sliding (ratio of resisting forces to driving forces) is calculated as:

Resisting Forces, $R = (N + F_A) \tan \phi$

Driving Forces, $D = T$

$$\text{FS} = R / D$$

If the anchors are not post-tensioned and locked-off at the specified design load, the soil friction angle should be based on residual strength, since the anchors will not fully develop their capacity until the slope has begun to move. It should be noted that the anchor force, F_A , could represent one anchor per square metre with a design capacity of 10kN, or five anchors per square metre, each with a design capacity of 2kN. The number of anchors per square metre is determined by the design engineer.

Similarly, for the case where the soil has cohesion and/or seepage forces present, the anchor force can be incorporated into the equations illustrated in Figure 7, below:

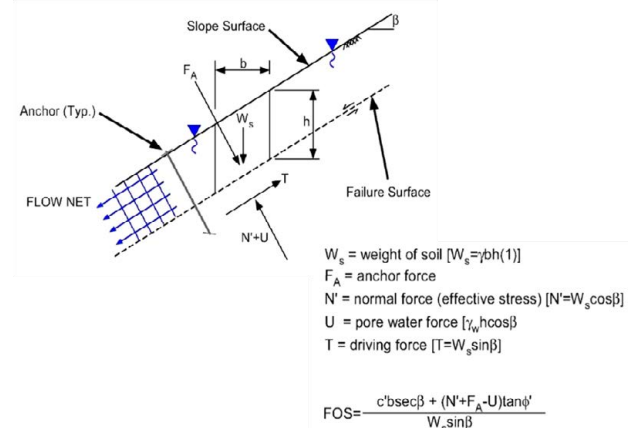


Figure 7. Infinite slope model calculation, with cohesion/seepage forces (Durham et al., 2012)

As for the case with cohesionless soil without seepage, if the anchors are not post-tensioned and locked-off at the specified design load the cohesion should be based on residual (fully softened) strength.

The anchor force (F_A) is controlled by the pull-out resistance of the anchor, which is a function of the anchor head dimensions, anchor length, geotechnical soil properties, and in situ moisture conditions. Pull-out resistance should be verified in the field with in-situ testing. A FS value ranging from 1.2 to 1.5 is recommended, and selection should be based on the quantity and quality of field testing and engineering judgment, the type of project, relevant codes and standards, consequence of failure, design life, etc. The anchor force F_A is limited by the punching shear capacity of the bearing plate through the HPTRM, which is a function of the plate material, plate dimensions, and the plate's mechanical interaction with the HPTRM.

5 ARVS SOLUTION FOR DEEP-SEATED ROTATIONAL FAILURE (LIMIT EQUILIBRIUM ANALYSIS)

Many larger slopes have complex geometry anticipated deep-seated rotational failure modes that do not lend themselves to the simplified assumptions of an infinite slope analysis. In these instances, conventional limit equilibrium analysis may be used to evaluate the anchorage design. Commercially available computer programs such as Slope/W or SLIDE have been used with successfully in modelling and designing ARVS solutions. SLIDE for example utilises several different limit equilibrium methods (such as Bishop, Spencer, Morgenstern-Price, Janbu, among others) for comparison of critical slip surfaces with the lowest FS values. The program allows the user to employ a circular surface grid search to iterate to the critical slip circle in a rotational sliding failure (or potential failure). The software is also effective in analysing non-circular sliding failures (or potential failures) consistent with the infinite slope model for translational slope movement (block glide), and cases where earth materials are heterogeneous and/or anisotropic.

In utilising limit equilibrium analysis for ARVS design, HPTRM should be modelled as a reinforcement layer, more specifically as a geotextile (although it is not a geotextile) where the tensile strength of the material can be input into the support properties.

When modelling the HPTRM, the force application should be set to passive (a module allows the user to specify this value). This limits the HPTRM in developing a resisting force only after some slope movement has occurred. The strip coverage is typically set to 100 percent as there are generally no gaps between the layers of HPTRM.

When modelling the PDEAs, an "End-Anchored Support" option is typically chosen in the software. The force application for the PDEAs is generally set to active, since the anchors will exert a force upon the slope without any prior slope movement (where tensioned upon installation). If the anchors are not post-tensioned, then they should be treated as a passive force and modelled accordingly.

The example illustrated in Figure 8 below shows a proposed slope subject to stability analysis for failure

potential in a rotational sliding mode. In this scenario, a slope comprised of a non-homogeneous mixture of soil types was analysed for stability when altered to a steepness of 1H:1V. Both soil types identified in the slope had cohesion, but groundwater was not present and pore water pressure was not applicable. Using a trial-and-error method of reinforcement design, an ARVS was modelled using an HPTRM and PDEAs of sufficient anchor head, tendon, and bearing plate size to achieve a 9kN pull-out strength capacity (assumed, to be verified in the field through load testing). Spacing of the PDEAs was proposed on 0.9m centres in two directions. Based on these parameters, the SLIDE program completed a search for the radius of the critical slip surface and returned an FS of 1.3 from the Bishop Simplified Limit Equilibrium Method. To provide a more conservative design, the effects of established vegetation on increased shear strength of the soil mass are neglected. This is suggested to be considered as a design requirement in general for all projects as it may take time to establish vegetation and a failure could occur in the meantime.

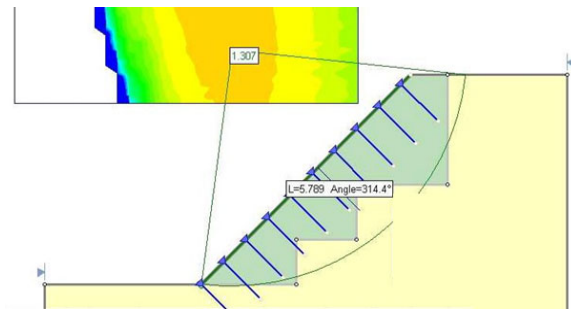


Figure 8. Example limit equilibrium analysis Modelled in SLIDE (Durham et al., 2012)

6 CONCLUSION

The Anchor Reinforced Vegetation System (ARVS) is a component system consisting of a High Performance Turf Reinforcement Mat (HPTRM) to provide erosion protection and surficial strength, coupled with Percussion Driven Earth Anchors (PDEAs) for resistance to shallow-plane instability. The system is designed to optimise rapid vegetation growth and keep soil in place, thereby resisting mobilisation of soil masses associated with sliding failures of slopes. In addition, the ARVS serves to isolate the underlying soil from erosive forces by providing continuous coverage of the slope face. The ARVS can be used as either a passive or active reinforcement system which makes this solution a unique solution. PDEAs are not only effective in resisting the shallow shear forces associated with both modes of failure, but they also force the potential rotational failure arcs deeper into the ground, thus producing a curved failure path with greater normal stress along the lower half.

Where the failure mode is expected to be a relatively shallow, surficial failure, a basic infinite slope analysis is often appropriate. Commercially available computer programs such as Slope/W or SLIDE have been used successfully in modelling and designing

ARVS solutions for deep-seated rotational failure analysis.

Key physical and material properties of the component system include optimal ultraviolet resistance, flexibility, and tensile strength of the HPTRM, along with its ability to promote vegetation establishment through increased soil and moisture retention.

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Sustainable Lining for Underground Hard Rock Openings

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ABSTRACT

Conventional cast-in-situ concrete linings are costly, time consuming and environmentally unfriendly solutions for supporting hard rock excavations, because concrete linings are unable to utilise the inherent strength of the rock. It is well understood that rock support is the application of a reactive force to the surface of an excavation such as concrete lining for example; whereas rock reinforcement is a means of conserving or improving the overall rockmass properties from within the rockmass by techniques such as rock bolts, cable bolts and ground anchors. This rock reinforcement strategy introduced to the tunnel roof and walls is considered to be a supporting element capable of sustaining a thrust at the arch ends. However, a potential small rock wedge failure is highly probable and may occur in between rock bolts. To deal with this issue, shotcrete linings will then be introduced acting as thin "protective skin" liners to support the rock surface with performance similar to a reinforced concrete slab. As the reinforced rock arch provides the required support to the opening, this shotcrete liner will then be designed to act as permanent protection cover and to comply with structural and durability requirements. Examples referenced in this paper demonstrate that the thickness of this shotcrete liner is relatively thinner than traditional concrete lining. Due to reduction of the permanent concrete lining thickness, the total excavation volume of caverns/tunnels is also reduced. As it minimises the use of cementitious products for permanent lining construction, thus leading to reduced CO₂ emissions and lower energy.

Keywords: excavation, failure, reinforcement, rockmass, support

1 INTRODUCTION

Other than mining, many of underground openings may be used for storage, transporting personnel, conveying materials and supplies, civil facilities, etc. Based on their function and service lifetime, these openings must be "sustainably" designed with an adequate factor of safety and constructed with proper work sequence.

In terms of sustainable solutions for openings in jointed hard rock, rock reinforcement approach is one of the underground rock-support methods and strategies. Rock reinforcement is a means of conserving or improving the overall rockmass properties from within the rockmass by techniques such as rock bolts, cable bolts and ground anchors. Rock support adopting conventional cast in-situ concrete lining is an expensive and time-consuming activity for rock caverns and tunnels and is considered inefficient for supporting hard rock excavations. Also, structural concrete is a mixture of cementitious products that require heat process in manufacturing and this process exhausts a huge volume of CO₂ to atmosphere to accelerate the greenhouse effect.

Hudson & Harrison (1997) and Kong & Garshol (2015) addressed that concrete linings are unable to utilise the inherent strength of the rock, particularly of strong rock with very poor quality of rockmass (i.e. $Q \geq 0.1$ or $GSI \geq 30$.) or better rock in terms of Q-value (Barton *et al.* 1974) or Geological Strength Index (Hoek, 1994; Hoek and Brown, 1997) respectively.

Detailed rock reinforcement design comprising rock bolts and shotcrete for underground openings has

been discussed by Barrett & McCreath (1995), Uotinen (2011), and Kong & Garshol (2015). Further discussion on the constructability of shotcrete rock reinforcement from a sustainability point of view, such as less spoil generated, less cementitious products to be used for permanent lining, etc, is given in the following.

2 OVERVIEW OF ROCK REINFORCEMENT AND REINFORCED ROCK ARCH

Lang (1961) developed one of the commonly adopted empirical design rules based on a range of laboratory, field and theoretical studies. This was used for pattern rock-bolting of permanent excavations during the construction of the Snowy Mountains Hydro-electric Scheme in Australia. These empirical design rules have been reviewed by Brown (1999) and later re-modelled by Hoek (2007) using patterned bolts to stress up aggregate in a frame model.

Based on Lang's (1961) findings [Figure 1(a)], Hoek's (2007) model demonstrated that a zone of compression is induced in the region shown in orange [Figure 1(b)]. This will provide effective reinforcement to the rockmass when the rock bolt spacing S is less than 3 times the average rock piece diameter, and the rock bolt length L will be approximately of $2S$ to $3S$ (particularly a rockmass with closely spaced joints). The "zone of compression" of rockmass has been treated to become isotropic in strength due to the effects of tensioned pattern bolts. An axial pre-stress is developed due to Poisson's effect of normal stress on account of the bolt's pre-tension. This pre-stress can stabilise the rock beam effectively as in the case of a pre-stressed continuous concrete beam,

which agrees with the Bischoff and Smart (1997) suggested model of reinforced rock arch. If a highly compressible feature such as a fault or a clay seam, or a highly fractured rockmass crosses the

compression ring, it is possible that the required compression could not effectively be developed, and that the reinforcement will be inadequate.

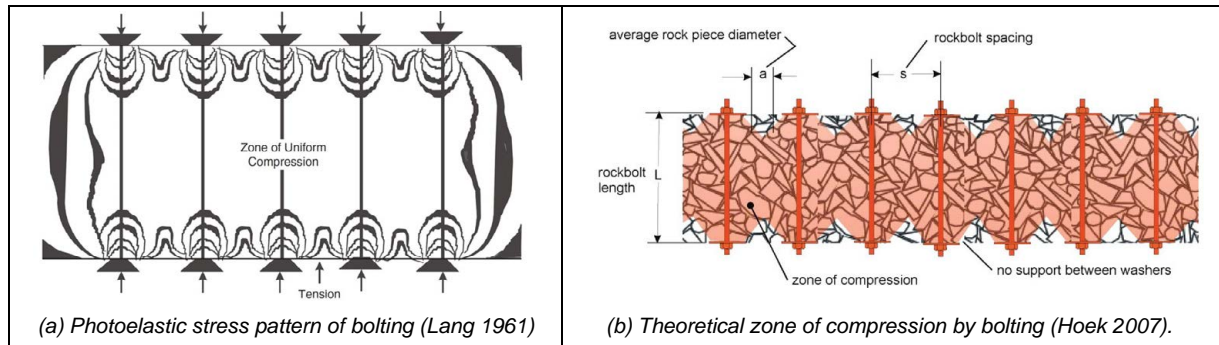


Figure 1. Illustration of "zone of compression" formed by stressed pattern bolt in media.

Bischoff & Smart (1997) and Kong & Garshol (2015) give detailed discussions on the concept of the reinforced rock arch. This concept is based on improving the strength of the rockmass at the tunnel walls by application of confining pressure via the bolts. The rock arch formed by the tunnel walls is considered to be a supporting arch capable of sustaining a thrust at the arch ends as shown in Figure 2.

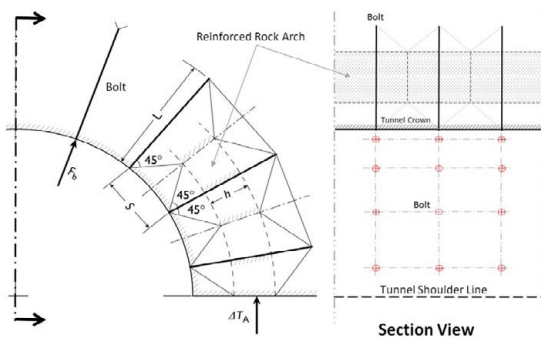


Figure 2. Geometry of tunnel bolting and rock arch thickness.

The Bischoff and Smart (1997) model showed that the thrust capacity of the rock arch is simply determined by the ratio of unconfined compressive strength to tensile strength with the provided bolt pressure. The thrust capacity of the rock arch (ΔT_A , half span of opening, in kN) is expressed as:

$$\Delta T_A = \frac{\sigma_c}{\sigma_t} \times \frac{F_b}{S_2} (L - S) \quad [1]$$

Where: ΔT_A is thrust capacity of the rock arch; h is reinforced rock arch depth; S is rock bolt spacing; L is rock bolt length; F_b is bolt force; σ_c is the unconfined compressive strength (UCS) of rockmass; and σ_t is the tensile strength of rockmass.

The thrust capacity of the rock arch (ΔT_A) can be equivalent to the Q support pressure (Barton *et al.*, 1974). According to a Mohr-Coulomb criterion, (σ_c/σ_t) can be rewritten as 'q':

$$q = \tan^2\left(45 + \frac{\phi}{2}\right) \quad [2]$$

Where ϕ is a friction angle (in degrees) of the rockmass. Based on the equations [1] and [2], the bolt length and spacing is entirely controlled by the UCS and tensile strength of the rockmass around the opening. In terms of rock strength, a stronger rockmass may require shorter bolt lengths and contrarily a weaker rockmass may need longer bolt lengths in place.

Comparing the Eq. (1) to Q-bolt length equation [i.e. $L_{Q\text{-system}} = 2 + (0.15B/ESR)$], as shown in Barton *et al.* 1974], it is hard to get agreement or a close relationship to determine the design bolt length using both approaches. For most cases when adopting closer bolt spacing, the estimated bolt length using Eq. (1) is shorter than the Q-bolt length. Contrarily in a wider bolt spacing arrangement, the estimated bolt length using Eq. (1) is longer than the Q-bolt length. With confidence of more than 40 years' practical experience using the Q-system and the above findings of the bolt length relationship, it is recommended the design bolt length should be determined using both approaches, whichever longer bolt length is estimated.

3 DESIGN OF SHOTCRETE LINER WITH ROCK REINFORCEMENT CONSIDERATIONS

With the effect of tensioned pattern rock bolts forming a reinforced rock arch, shotcrete lining for underground openings can be considered as a thin shell liner. It is called "Shotcrete Rock Reinforcement" (SRR). For this thin shell shotcrete liner, six failure modes have been described by Barrett & McCreath (1995) comprising adhesive, flexural, shear, punching, compressive and tensile failures as illustrated in Figure 3. Key findings from their study revealed that falling block tests indicated the most likely failure modes would be adhesive, shear and punching.

Lang (1961), Bischoff and Smart (1997) and Hoek (2007) concluded that a possible contribution load

acting on the shotcrete liner between bolts would be considered a pyramid shaped wedge, i.e. for the worst case scenario in highly fractured rockmass, it is 45° projected from the base plate of the rock bolt (see Figure 4 for illustration). Bischoff and Smart's model specified that the rockbolt base plate size will be one of the key elements in controlling the size of potential failure wedge forming between bolts.

Barrett & McCreath (1995), Uotinen (2011), and Kong & Garshol (2015) gave full discussion on the design checks against adhesive, flexure, shear and punching failures of shotcrete liners using Eurocode 2 (BSI, 2004); and Christine *et al.* (2017) has also conducted a revisited shotcrete support design in blocky ground. Discussion on the design of shotcrete layers is not given in this paper (refer to the captioned references).

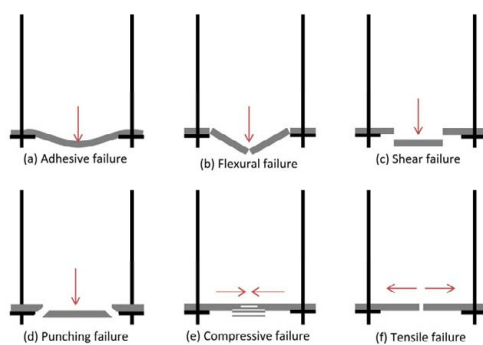


Figure 3. Failure modes of shotcrete between rock bolts (excerpted from Kong & Garshol, 2015).

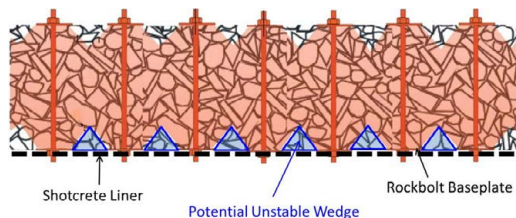


Figure 4. Schematic layout of SRR support measure.

Table 1: SRR calculated values (shotcrete) and Phase² model results (excerpted from Kong, 2018).

Model ID	Shotcrete Thickness (mm)	Phase ² Results			SRR Calculated Shotcrete Capacity	
		Max. Shear Force (MN)	Max. Bending Moment (kNm)	Max. Boundary Deformation (mm)	Permissible Shear Force (MN)	Permissible Bending Moment (kNm)
Q10	77	0.047	1.0	3.5	0.280	4.816
Q5	57	0.028	2.0	17	0.185	2.674
Q2.5	59	0.024	0.787	26	0.17	2.861
Q1	39	0.013	0.339	32	0.097	1.266
Q0.4	27	0.012	0.336	57	0.057	0.612
Q0.1	25	0.017	0.247	70	0.042	0.525

5 CONSTRUCTABILITY OF SRR AND CASE EXAMPLES

5.1 Durability of Rock Bolts

Regarding the durability of permanent rock bolts with single corrosion protection of hot-dip galvanised high-yield or mild steel bars, an allowance for generalised corrosion of 2 mm

4 ENGINEERING RESPONSE OF SRR AND DEFORMATION OF OPENINGS

Kong (2108) carried out a study using SRR approach to investigate a range of rockmass qualities from $Q = 0.1$ to $Q = 10$, with rock strength (UCS) of 70 MPa, and with the underground opening span as 24 m, at 14.8 m in height with rock cover of 45 m. The results of the findings of his investigation is summarised in Table 1.

The result of Phase² models show that all six models are stable at the opening and no yielded bolt and shotcrete liner are found. In different rockmass qualities, a boundary deformation of the opening is modelled ranging from 3.5 mm (in Q10 model) to 70 mm (in Q0.1 model). A percentage ratio of boundary deformation to the span of opening varies from 0.015% (in Q10 model) to less than 0.292% (in Q0.1 model). A summary of the model results is presented in Table 1.

In theory, this magnitude of deformation is less than the EC2 (BSI, 2004) or AS3600 (SAL, 2018) requirements of 'span/250'. In one of the examples, a 62 m span Norwegian Olympic Ice Hockey Cavern (Barton *et al.*, 1994) adopted rock reinforcement techniques and reported that the cavern roof deformation of about 10 mm was recorded for the section of rock quality $Q = 1$, and the final shotcrete liner thickness was 50 mm.

For a range of different rockmass qualities, the results of SRR with numerical modelling are in close agreement to Mahar *et al.* (1975) and the Hoek *et al.* (2000) study which collected various rules of thumb from a variety of sources and included them in their monograph. These empirical rules of thumb prescribe that the shotcrete thickness for underground openings in jointed rockmasses and under various in-situ stress conditions, range from 25 mm to 100 mm.

sacrificial thickness on the radius is to be made (GEO, 1984). Hence the determination of design working load of bolts should be considered as based on a bolt diameter minus 4 mm. Where grout is used, a minimum cover of 6 mm should be provided to the bolt (GEO, 1984). However, Pells & Bertuzzi (1999) suggested a minimum grout cover to the bolt should be 10mm. The free length of the

bolt should also consist of grout, grease-filled sheathe, or other suitable protection method.

SINTEF (1999 & 2000) undertook evaluation of the accelerated corrosion tests for the CT-Bolt (one of the commercially available self-anchorage rockbolts) which is coated with a specific combination of hot-dip galvanising, zinc phosphate and powder coating. The evaluation showed that the bolt is capable to sustain lifetimes up to 150 years. This treatment gives unique protection against corrosion and wear and tear. Singapore's Jurong Hydrocarbon Storage Caverns used the CT-Bolt as a permanent rock reinforcement measure.

5.2 Waterproofing

One of the structural integrity problems presented at cavern/tunnel permanent linings is water seepage. Commonly, sheet waterproofing membrane is adopted in cast in-situ concrete lining for caverns/tunnels. In view of the physical nature of permanent shotcrete lining, it is possible to use spray-applied waterproofing membrane. Su (2013) found that spray-applied waterproofing membrane demonstrated that there is sufficient and reliable tensile and shear strength at the shotcrete-membrane interface to prevent water permeating through cracks and seeping along the interface in such a way that it could load the permanent lining (i.e. the permanent shotcrete liner) directly. ITatech (2013) gave further guidance on the design for spray-applied waterproofing membranes for underground structures. As reported by Wallis (2007) and Garshol & Lacerda (2007), a 30-year old Chekka highway tunnel north of Beirut was experiencing water ingress through cracks in the existing concrete lining. Tunnel lining repair works without demolition of the existing lining had been carried out successfully using spray-applied waterproofing membrane with a 4 cm thick finish of steel-fibre reinforced shotcrete.

In addition to spray-applied waterproofing membranes, modern concrete technology (ACI, 2016) has proven that permeability-reducing admixtures such as crystalline products (CP) and hydrophobic pore blocker(s) (HPBs) are capable of improving water absorption and permeability effectively if they are added to concrete. Some permeability-reducing admixtures (e.g. CP and HPBs) report to have two distinct actions. The first is a reaction of the hydrophobic compounds with hydrating cement phases, modifying the cement paste matrix and changing the surface tension of the capillary surfaces, reducing the capillary movement of water through the concrete mass. The second action of the HPB is reported to collect in the capillaries forming a physical plug, blocking the capillary system and preventing further water entry (Ramachandran, 1995; Aldred, 1988).

These permeability-reducing admixtures in concrete have been reported to resist water penetration (with coefficient of permeability less than 1×10^{-12} m/s) against hydrostatic pressure (British Board of Agrément, 2006 and ACI, 2016). Shotcrete treated

with some permeability-reducing admixtures (e.g. CP) has been used successfully for rehabilitation of the Ohio's National Road Bikeway Tunnel (Penetron, 2013). The tunnel was reported to have water leakage and a dangerous ice build-up, which threatened use of the tunnel.

5.3 Fireproofing

Without a doubt, rock is a very fire-resistant material and hence there are insignificant fire-induced effects to tensioned rock bolts that would cause the loss of reinforced rock arches. Only an element of the rock bolt head needs to be provided with fire proofing cover. With reference to Eurocode 2 (BSI, 2004) for example, a minimum shotcrete cover of 70 mm to steel reinforcement should be provided to avoid falling-off of concrete (or shotcrete) in the latter stage of fire exposure. Thus, the schematic illustration of shotcrete fire protection cover to the rock bolt head is shown in Figure 5.

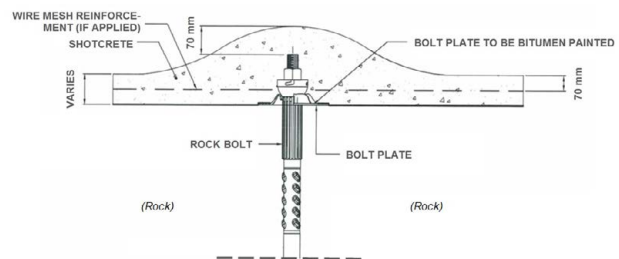


Figure 5. Schematic illustration of shotcrete fire protection cover to rock bolt head.

In case of fire, spalling of shotcrete can be prevented by adding polypropylene micro fibres to the shotcrete mix as an additional fire protection layer. The effect is achieved by melting of the fibres at low temperature, leaving open channels in the concrete that will allow steam to evacuate without building internal splitting pressure within the concrete.

5.4 Reduction in Excavation Volume

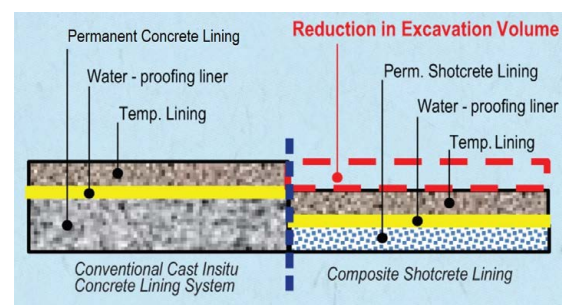


Figure 6. Illustration of cost saving and less excavation by using shotcrete rock reinforcement.

For the SRR support and construction method, the use of rockbolt and shotcrete in lieu of in-situ concrete as secondary (permanent) lining is adopted. This means that the excavation envelope (i.e. volume) can be reduced by about 10% (depending upon the cavern/tunnel size and length) (see Figure 6 above), and concrete volume reduced

by up to 50% due to the required thickness of the permanent shotcrete liner being much thinner than the structural concrete lining support. With less use of cementitious products, this leads to a reduction in the carbon footprint. As shotcrete can be sprayed any time after excavation, there is no need to erect formwork, pour concrete and strip shutters, which gives program advantages as well.

5.5 Case Examples of Using SRR

Over the past 30 years, a number of underground excavation projects have adopted rock reinforcement with shotcrete liners for ground support. A list of selected projects using SRR is summarised in Table 2.

Table 2: Selected case examples using SRR.

Country	Project	Year	Cavern Size and Details
Norway	Olympic Ice Hockey Cavern	1994	<ul style="list-style-type: none"> 62 m span Norwegian Olympic Ice Hockey Cavern, 25-50m ground cover (Barton <i>et al.</i>, 1994). Alternate 6m long Ø25mm rockbolt and 12m long twin-stranded cable bolt (Ø12.5mm) at 2.5m and 5m centre-to-centre; 50-100mm thick permanent shotcrete lining.
Hong Kong	Saltwater Service Reservoir Caverns	2009	<ul style="list-style-type: none"> Cavern size with 17m span, 15m high, min. ground cover 45m, (Chung, 2011). 3mm thick of spray-on waterproof membrane with permanent steel fibre shotcrete liner applied to the cavern roof.
Singapore	Jurong Hydrocarbon Storage Caverns	2012	<ul style="list-style-type: none"> Cavern size typically 20m wide, 27m high constructed below sea level. (Zhou <i>et al.</i>, 2017). 4.4m long CT-bolts with steel fibre shotcrete liner of 60mm – 140mm in thickness.

6 DISCUSSIONS AND CONCLUSIONS

As discussed by Kong (2018), in most of the cases the design of rock bolt support for underground hard rock openings overlooks the rock bolt's function acting as reinforcement to form an in-situ rock arch. This rock arch that is formed by application of confining pressure via the bolts, utilises the inherent strength of the rock to support the rock load above the opening. With confidence of more than 40 years' practical experience using the Q-system, the support rock load using Q support load is feasible for the SRR approach. As the rock bolts have been designed to support the required rock load, the shotcrete lining may only be designed to retain a potential wedge load forming in between bolts.

Shotcrete linings (with or without reinforcement) can be designed in accordance with the available design codes and standards such as AS3600 (SAL, 2018) or Eurocode 2 (BSI, 2004) by considering thin "protective/supportive skin" liners to support the rock surface between rock bolts with performance similar to a reinforced concrete slab. Attention is drawn to the fact that the shotcrete liner cannot prevent deformation from taking place, especially in high stress environments, but it can assist in controlling deformation, particularly when used in combination with rockbolts, dowels or cables. Nevertheless, shotcrete support becomes very effective after rock bolt installations are carried out. Therefore, the approach of SRR with the patterned rock bolts and high-quality shotcrete provides one of the most stabilisation measures for underground openings.

The SRR approach may give an optimised solution in the thickness of shotcrete liner as a permanent structure for hard rock underground openings. In terms of constructability, as referred to in ITA (2010), a minimum of 60 mm thickness is recommended for permanent lining.

In evidence of accelerated corrosion tests, rock bolts with a combination of hot-dip galvanising, zinc phosphate, and powder coating has indicated lifetimes up to 150 years. Carbon steel bolts, cement grouted in an open-ended plastic sheath are acceptable for 100-year design life in which cement grout is considered to be part of the corrosion protection, if it is within a plastic sheath. Hence, the durability of rock bolts with design lifetime of 100 years, which complies with the Eurocode 0 (BSI, 2005), is feasible.

Composite shotcrete liner sandwiched with spray-applied waterproofing membrane or the use of permeability-reducing admixtures in the shotcrete mix are able to provide a one-piece waterproofing structural liner to prevent water permeating through cracks and seeping along the interface of the permanent cavern/tunnel lining.

Modern shotcrete technology reveals that the durability of shotcrete is governed by the same parameters as cast in-situ concrete. For quality and workmanship control to shotcrete, it is possible to follow the Specification for Tunnelling (BTS & ICE, 2010). In the case of fire, spalling of shotcrete can be prevented by adding polypropylene micro fibres to the concrete mix. The effect is achieved by melting of the fibres at low temperature, leaving open channels in the concrete that will allow steam to evacuate without building internal splitting pressure within the concrete.

It is concluded that the SRR for underground excavations, particularly of large span caverns constructed in non-squeezing hard rock media is able to provide sustainable and cost-effective solutions. Cost-wise, SRR reduces in construction time and resources that no formwork or rebar fixing are required for permanent lining, and less excavation volume is required. In addition, due to the reduction of the permanent concrete lining

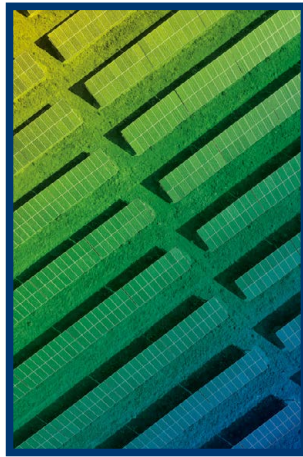
thickness, the total excavation volume of caverns/tunnels is also reduced. As it minimises the use of cementitious products for permanent lining construction, it leads to reduced CO₂ emissions and lower embodied energy.

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