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AUSTRALIAN GEOMECHANICS

JOURNAL AND NEWS OF THE AUSTRALIAN GEOMECHANICS SOCIETY

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VOLUME 61: NO.2 JUNE 2026

INCLUDING

Rebuilding confidence in construction materials testing: a pathway to integrity, innovation and industry renewal

Rethinking earthworks quality testing

A symmetrical tale

Embedded retaining wall design in accordance with Australian design standard AS5100.3-2017

Notes on high horizontal stress generation and post processing for deep excavation shoring wall using program Plaxis 2D

Redevelopment of heritage buildings in Sydney: case history of complex underpinning and support works

Soft soil stabilisation using admixtures from various solid waste materials

Structural controls of the Otway Ranges and hazards for road users

AUSTRALIAN GEOMECHANICS SOCIETY

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Cover image: Heavy Haul Rail, Pilbara, Western Australia
Photography and story: Robin Power

Ballast condition is a significant contributor to rail track performance, as is the condition of the supporting formation. Traditionally, an intrusive and costly process called 'pot holing' is utilised to gather data for planning rail track maintenance and renewal programs, providing rehabilitation works design inputs and decision making. Excavators are used to remove ballast at the sleeper ends of 2-3 sleepers, exposing the ballast profile. The depth and condition of the ballast is photographed, measured and material samples maybe taken for laboratory analysis. Then, a traditional DCP is used to test into the formation, capturing blows per 10 cm, which is often converted to CBR. The ballast is then scraped back into location. The process is typically repeated at x m intervals. A line shut is required with all the costs, planning and other implications associated with this.

An alternative approach used is a coupling of cone resistance vs depth profile with very high quality depth indexed down the hole imagery (video & photos). Rapid image analysis using AI including estimation of the soil grain size distribution (PSD) and water content allows a qualitative characterisation of soil. This is achieved using a Variable Energy Dynamic Penetrometer (VEDP) coupled with very high end camera technology and analysis software. This provides a non-destructive rail track ballast assessment (ballast fouling and ballast recovery rates) and condition monitoring of the formation (rail track substructure layers), all captured, processed and presented in digital environment. The technique is also used to calibrate Ground Penetrating Radar (GPR) data and to provide more information in problem areas.

AUSTRALIAN GEOMECHANICS SOCIETY

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VIEW FROM THE CHAIR

JUNE 2026

It is both an honour and a privilege to take on the role of National Chair of the Australian Geomechanics Society (AGS). I would like to acknowledge the significant contribution of my predecessor, Tim Thompson, and the Board, whose leadership has positioned the AGS strongly at a time of both opportunity and change.

The past two years have marked a significant transition for the AGS. The move to a company limited by guarantee has established a modern governance framework and strengthened our organisational foundations. Importantly, this transition is not merely administrative; it enables AGS to operate with greater clarity, accountability, and long-term sustainability.

This strengthened foundation comes at an important time for our profession. The broader environment in which geotechnical engineering operates is evolving rapidly. Across Australia, we are seeing the impacts of climate variability emerge more directly in our work through more intense rainfall events, prolonged dry periods, and increasingly complex ground responses. These are no longer abstract risks. They are influencing how we design for long-term performance.

Recent infrastructure projects continue to remind us that, despite advances in investigation and analysis, subsurface uncertainty remains one of the most significant risks facing our profession. The “unknowns” in the ground are still very real, and they continue to test both our assumptions and our designs. At the same time, expectations surrounding net-zero infrastructure are pushing the profession to think differently about material use, ground improvement, and efficiency, not only in terms of cost and performance, but also carbon outcomes.

Alongside these environmental pressures, recent global disruptions, particularly those relating to energy supply and fuel security, have further highlighted how dependent we are on factors extending well beyond the site boundary. For our profession, the lesson extends beyond energy alone; it is fundamentally about resilience and uncertainty. The conditions we design for, whether in the ground or within broader systems, are not fixed. Perhaps most importantly, recent global events have reinforced that uncertainty is not an occasional challenge, but a constant condition. They have also reminded us of a simple but important lesson: we must become better at recognising early warning signs before they develop into major failures.

In geotechnical engineering, this principle is particularly relevant. Whether dealing with slope instability, foundation performance, excavation movements, or embankment behaviour, failures are often preceded by subtle indicators. The challenge is not only collecting data but developing systems capable of converting data into timely decisions. This requires better baseline information, targeted instrumentation, regular monitoring and data collection, clear trigger levels, and, critically, the discipline to act when early signs emerge.

It is against this backdrop that the Australian Geomechanics Society is progressing development of the **2026–2030 Strategic Plan**, a major milestone for AGS. The strategy is being co-designed with input from the Board, Chapters, and previous national chairs, and will help define our purpose, priorities, and measures of success over the coming years. At its core, the strategy reinforces a simple but powerful objective:

To connect, empower, promote, and represent a thriving Australian geomechanics community.

While this objective may sound straightforward, it is intentionally focused. As a learned society, we cannot do everything. Our responsibility is to concentrate effort where AGS can provide the greatest value, whether through technical leadership, knowledge sharing, advocacy, or supporting the next generation of engineers.

The emerging strategic framework is structured around three key pillars: **LEAD, DELIVER**, and **GROW**. Early priorities include strengthening member value and engagement, advancing technical leadership and knowledge sharing, increasing advocacy and industry influence, and supporting sustainable organisational capability. Importantly, the strategy reinforces that AGS must remain **member-centred**, **focused**, and **purposeful**. Ultimately, however, the success of the strategy will depend on people, our volunteers, Board members, and contributors across all Chapters.

A key enabler of this strategy is the AGS’s ongoing digital transformation. Work is currently underway to develop an **AGS App** that will enhance how members engage with events, conferences, and technical content. This initiative is being aligned with broader improvements to our website and digital systems to create a more integrated and accessible member experience.

In parallel, discussions are progressing on a nationalisation of the Queensland Geotechnical Database, an initiative with the potential to improve how geotechnical data is shared and utilised across the industry. While still evolving, the long-term vision is for AGS to play an active role in supporting this national resource in collaboration with government partners.

Together, these initiatives represent an important shift for the AGS from being primarily a knowledge-sharing forum to becoming an enabler of data-driven practice and digital collaboration.

This evolution is particularly important given the rapid integration of artificial intelligence and data-driven methods into geotechnical practice. Advances in digital monitoring, remote sensing, and machine learning are enabling engineers to move from periodic assessment toward continuous, real-time understanding of ground behaviour. Today, AI and real-time data are revolutionising the way we interpret information, anticipate outcomes, and make decisions. For example, a new AI-powered landslide warning system developed in Hong Kong is expected to deliver real-time predictions of slope failure with accuracy exceeding 90%, using millions of historical data points and continuously learning from new information. Similarly, Auckland Transport has employed AI to map landslide risks across its road network through a GIS-based system analysing more than 30 contributing factors, including data from previous storm events.

These technologies are not replacing engineering judgement; rather, they are enhancing it. They allow us to better quantify uncertainty, detect early warning signs, and make more informed decisions across the lifecycle of infrastructure. Put simply, the greater the uncertainty, the greater the value delivered by data, monitoring, and intelligent analysis.

In many respects, the direction for our profession is clear. As engineering challenges become more complex and less predictable, the value of geotechnical expertise only increases. The role of AGS is not simply to respond to these changes, but to help lead how the profession adapts.

As the challenges facing our profession become increasingly complex and interconnected, collaboration remains central to the role of AGS. Addressing these emerging challenges will require not only technical innovation, but also stronger collaboration across industry, academia, and international partners. A strong example is our ongoing partnership with the New Zealand Geotechnical Society (NZGS) in updating the AGS (2007) Guidelines for Landslide Risk Management. This collaboration includes a formal memorandum of understanding and shared technical contributions across both countries. AGS is proud to support this effort as a project partner, helping to bring the updated guidelines to completion while ensuring its relevance and accessibility for practitioners in Australia. The AGS has also recently confirmed partial financial support for a series of NZGS Slope Stability Design Manuals in development. Importantly, these initiatives reinforce the value of regional and international collaboration. The challenges we face are rarely unique to one country, and there is significant benefit in working together to develop practical, evidence-based guidance. This is precisely where AGS can add meaningful value, not by duplicating effort, but by partnering with others to deliver high-quality outcomes for the profession.

Looking ahead, the AGS has a strong pipeline of initiatives, including continued development of technical guidelines and publications, expansion of education and training programs across regions, ongoing digital transformation initiatives, support for major international conferences and events, and increased focus on outreach, diversity, and inclusion. At the same time, we must remain focused on our core purpose: supporting a thriving geomechanics community. This requires balancing growth with technical excellence, and innovation with the practical needs of our members.

The strength of the Australian Geomechanics Society has always been its members, their expertise, their willingness to contribute, and their commitment to advancing the profession. As we move into this next phase, I encourage all members to engage actively with the AGS, whether through technical contributions, mentoring, or participation in events and initiatives.

I look forward to working with you all as we continue to build on this strong foundation and help shape the future of geomechanics in Australia.

Amir Shahkolahi

National Chair, Australian Geomechanics Society

WOMEN IN AGS (WIAGS)

IWD 2026: AGS Celebrates International Women's Day

Sunday 8th of March marked International Women's Day, which is celebrated in various ways around the world. A number of countries such as Cambodia, Kyrgystan, Mongolia, and Zambia declare it a public holiday for all. In China and Madagascar is it a holiday for women only. Here at the AGS we found various ways to celebrate the day across the chapters.

Brisbane – Walk and Talk

Kicking off on Monday 9 March, the Queensland Chapter held a Walk + Talk event. The wet weather didn't dampen the spirits of the 30 women and men who enjoyed a leisurely walk by the Brisbane River, stopping by some local landmarks for a photo and heading over the new(ish) Neville Bonner Bridge for a different view of the city. I find the walking events a great and natural opportunity to chat and meet new people, rather than being trapped in a meeting room juggling a plate and a drink at a more traditional networking event.

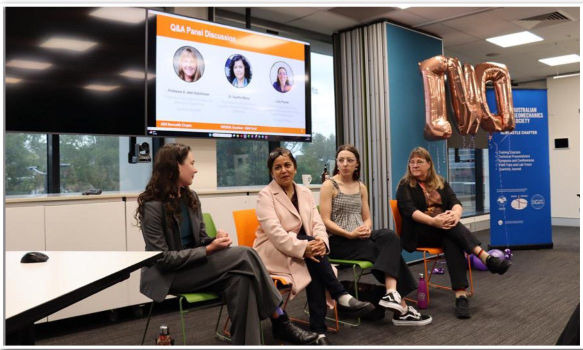
Walkers at the "Brisbane Sign" by the river on a rainy day

Following the walk, we assembled back at the Jacobs office for breakfast and a chat with Esnart Kazhingu from Mamuke in Zambia. Mamuke was our charity partner for the morning's event with a portion of ticket sales being donated to their work educating vulnerable children.

Natalie Quinlisk interviewing Esnart Kazhingu

Newcastle – Panel and Breakfast

Later that week, our Newcastle Chapter hosted a breakfast and panel event at the local GHD office. During the Q&A panel discussion, Professor D. Jean Hutchinson (Professor Emerita of Geological Engineering - Queen's University, Canada), Dr. Sujatha Manoj (Professional civil/geotechnical engineer with over 33 years' experience) and Lucy Poyser (Undergraduate geotechnical engineer - Douglas Partners, Newcastle) shared their unique experiences of working in geomechanics. We celebrated their achievements and discussed the ongoing challenges faced by women in our field, exploring what can be done to further reduce gender disparity (especially in senior positions) and help #BalanceTheScales.



Newcastle Panel (L-R) Abigail Watman, Dr Sujatha Manoj, Lucy Poyser and Professor D. Jean Hutchinson

Adelaide – Walk and Talk

Later in March, The SA/NT Chapter hosted an IWD Walk and Talk event, providing an informal setting for members to connect outside the traditional technical forum. The event encouraged discussion and reflection on inclusion, professional experiences, and career pathways within the ground engineering community. The relaxed nature of the Walk and Talk allowed for attendees to make many meaningful connections, strengthening the WIAGS community across a range of career stages, and reinforcing the Chapter's commitment to fostering a welcoming and inclusive professional environment.

The group of walkers in Adelaide

Melbourne – Picnic Lunch in the Gardens

The Victorian Chapter hosted a relaxed and engaging (slightly belated) International Women's Day gathering that brought together members of the geotechnical community in an informal, welcoming setting. Held at the Flagstaff Gardens picnic area, the BYO lunch format encouraged a casual atmosphere where attendees could connect, share experiences, and strengthen professional networks across academia and industry.

Conversations ranged from Melbourne weather to career development and mentorship and technical challenges! We hope to have another event soon to continue engaging with our community and promoting women in the profession.

Lunchtime in Melbourne

Supporting the WIAGS Mission

The overarching intent of these events is to increase women's engagement with the AGS community. The panel events give us the opportunity to celebrate specific women's successes, and the networking events help to create an inclusive culture. All of this aligns with our WIAGS mission:

"Increase women's engagement in the AGS community, celebrate their successes and foster an inclusive culture that attracts and retains women."

Thank you to all who attended and to each local WIAGS Committee for making these events happen.

Many chapters will be hosting events in celebration of International Women in Engineering Day this coming June. We appreciate the support of the wider AGS community at these events.

By Natalie Quinlisk, Chair of WIAGS National Committee

CHAPTER NEWS

NEW SOUTH WALES

63rd Rankine Lecture – From Geo-monitor to Geo-adapt
Event Mentors: Asal Bidarmaghz (UNSW)

Professor Kenichi Soga's 63rd Rankine Lecture, "From Geo-monitor to Geo-adapt", was held by the Australian Geomechanics Society on 23 February 2026. The lecture presented a clear shift in geotechnical engineering: from simply monitoring infrastructure to using monitoring data to actively improve design, construction and long-term asset management. His main argument was that geotechnical systems should no longer be treated as static assets designed once and checked occasionally. Instead, tunnels, pipelines, foundations and other ground infrastructure can be managed as adaptive systems, where distributed sensing, fibre-optic monitoring, wireless sensors and data analytics provide continuous feedback on real performance.

The lecture showed how high-resolution monitoring can improve understanding of ground–structure interaction, construction response, strain development and long-term deterioration. For tunnels, this means better tracking of deformation and lining behaviour. For pipelines, it allows early detection of localised ground movement and strain demand. For deep foundations, it provides stronger evidence on load transfer, settlement and construction quality. The key value is not the sensor itself, but the ability to convert data into engineering decisions.

Overall, Soga argued for a future where geotechnical design becomes more performance-based, data-informed and adaptive. This is directly relevant to climate-responsive infrastructure and energy geostructures, where piles, tunnels and retaining walls may need to be monitored and managed over their full life under changing thermal, hydraulic and mechanical conditions.

63rd Rankine Lecturer Professor Kenichi Soga presenting at AGS Sydney.

AGS 2026 Industry Day at Western Sydney University
Event Mentors: Pan Hu (WSU) and Asal Bidarmaghz (UNSW)

The AGS 2026 Industry Day at Western Sydney University was held on 30 March 2026 at the Peter Shergold Building, WSU, Parramatta. Hosted by the Sydney Chapter of the Australian Geomechanics Society, in collaboration with Western Sydney University, the event introduced undergraduate, postgraduate and research students to geotechnical engineering, engineering geology and AGS activities. The programme featured short presentations by practitioners, including Dr Naveen Meena from Beca, Aidan McKenzie from Transport for NSW, Samantha Ross from GHD, Manuel Neves and Bethan Murrant from Fortify Geotech. The speakers shared practical insights into career pathways and day-to-day professional practice across soil–structure interaction, slope stability, geological modelling, site investigation, infrastructure delivery, foundation engineering, retaining structures, ground improvement, and construction-phase geotechnical advice. Overall, the event strengthened links between students, academia and industry, while promoting AGS engagement and professional development for the next generation of geotechnical engineers and engineering geologists.

AGS Industry Day presenters from left to right: Dr Naveen Meena, Aidan McKenzie, Samantha Ross, Manuel Neves and Bethan Murrant.

Honorary Life Member Presentation: "Compaction, can it be Intelligent?"
Event Mentors: Adnan Sahyouni (Menard)

The Honorary Life Member presentation by Professor David Airey from the University of Sydney, titled "Compaction, can it be Intelligent?", was held on 8 April 2026. The event attracted 111 registrations from across industry and academia, bringing together participants from different professional backgrounds, experience levels and age groups.

Professor Airey revisited the fundamentals of soil compaction and challenged the profession to modernise current compaction quality assurance and quality control practices. A central message was that compaction still relies heavily on long-standing empirical methods, while

Professor David Airey presenting to the AGS Sydney Chapter.

testing, interpretation and standards have not evolved at the same pace as other areas of engineering and science. The presentation highlighted the limitations of density-only compliance and reinforced the need to better account for moisture content, degree of saturation, stiffness and long-term performance.

The lecture covered intelligent compaction, stiffness-based measures, field testing and future directions for QA/QC. Professor Airey discussed the value of GPS coverage, pass-count data, layer thickness, machine operation and processed compaction measurement values in improving process assurance. However, he also noted that truly intelligent compaction systems are not yet fully available and will require stronger interpretation frameworks, better field data and potentially AI or machine-learning tools.

A field trial example from the Toowoomba Range Second Crossing was used to compare stiffness measures, layer thickness and pass-count effects across different aggregate and rock materials. The discussion showed that stiffness-based indices can be highly sensitive to material variability, reinforcing the importance of good stockpile management, moisture control and reliable moisture content data.

Full room – showing the attendees.

QUEENSLAND

This report includes the Queensland Chapter news from March to April 2026. Chapter activities included multiple evening technical presentations, a morning social event, and a regional lecture series.

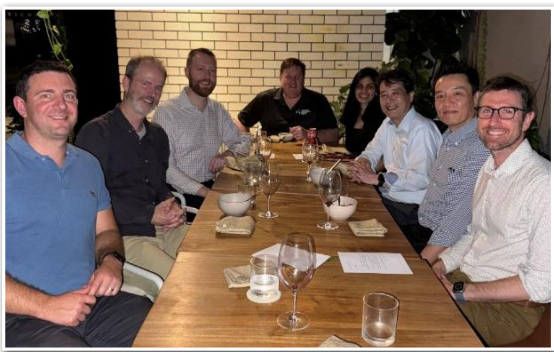
63rd Rankine Lecture – Kenichi Soga
From Geo-monitor to Geo-adapt: leveraging distributed sensing and data analytics for performance-based design, construction, and maintenance

Thursday 5 March 2025

Over 60 members and guests gathered at Rydges Fortitude Valley for an evening lecture featuring Prof. Kenichi Soga, who delivered his Rankine Lecture on distributed sensing and data analytics across a wide range of geotechnical applications.

Kenichi Soga was an engaging and insightful speaker, sharing examples that demonstrated the growing impact of sensing technologies on geotechnical practice. He brings an exceptional academic background, currently serving at the University of California at Berkeley, and formerly at the University of Cambridge.

Kenichi Soga in Brisbane.



Speaker dinner, Left to Right: Matt Stewart, Tim Thompson, Jared Priddle, Jon Gibbs, Arsh Kaur, Kenichi Soga, Jun Sugawara, Vincent Blanchet.

International Women's Day: Walk + Talk

Monday 9 March 2026

It was a wet Monday morning, when we gathered 30 women and men in Brisbane for our annual AGS Walk + Talk event. We dawdled in the drizzle but enjoyed the warmth of conversation amongst friends new and old. The coffee was hot, the food was fresh, and the gift bags – gorgeous!

Our charity partner for the morning's event was Mamuke – acts of hope. We were honoured to welcome the founder, Mrs Esnart Kazhingu from Zambia, who shared about the strength of women and the importance of education. Mamuke run a preschool and nutrition program for vulnerable children in a very poor community. Did you know that in Zambia, International Women's Day is celebrated with a public holiday!?

Thank you to all who attended and the QLD WIAGS Committee for making it happen. Our thanks also go to Jacobs for generously sponsoring the venue.

The walkers braved the rain for a riverside stroll and chat.

Each attendee took home a small gift bag and memento of the morning.

Unlocking the Challenges of Deep Underground Construction

Wednesday 25 March 2026

Over 50 members and non-members gathered at Engineers Australia to listen to Dino Sarac from Bechtel. He shared lessons from a complex case study on constructing a deep underground station beside an existing 26metredeep basement, highlighting the challenges of supporting a high and narrow sand backfill with a complex retaining system of rock pillars and props.

It was a valuable practical presentation for anyone involved in deep excavations, geotechnical design, or construction monitoring, and a great reminder of the value of sharing realworld lessons from major projects.

Dino Sarac presenting at Engineers Australia.

Speaker dinner, Left to Right: Jaime Wilson, Dino Sarac, Vincent Blanchet.

2025 Peter Hollingsworth Honoured Lecture: Lessons from 44 Years of Change in Geotechnical Engineering

Thursday 23 April 2026

More than 100 members and guests convened at Sofitel Brisbane Central to hear Dr Burt Look OAM present his 2025 Peter Hollingsworth Honoured Lecture.

Burt delivered a reflective and philosophical talk on how geotechnical engineering has evolved through advances in site investigation, laboratory testing, data interpretation, and risk communication, particularly in the context of expansive clays, residual soils, and weathered rock common to Queensland. Case studies demonstrated how data noise and well-known cognitive biases in geotechnical engineering can systematically distort professional judgement. He emphasised the need to better integrate field observations, laboratory data, and reliability-based thinking, while challenging conservative assumptions and "judgement" driven practices that can obfuscate the real engineering risk. The lecture highlighted ongoing gaps in current practice and reinforced the importance of reliability-based transparency and accountability in modern geotechnical engineering.

Burt is highly regarded both nationally and internationally and is especially well-known in Queensland. His career has been predominantly in consultancy, where he has held senior technical and business leadership roles with Connell Wagner (Aurecon), SKM (Jacobs), and FSG.

The AGS QLD Chapter awards the Peter Hollingsworth Honoured Lecture every two years. Peter was a Queensland-trained civil engineer (1951) and licensed surveyor (1954) who played a formative role in Australian and Southeast Asian geotechnical engineering through innovative project delivery and environmental impact studies across a wide range of sectors.

Dr Burt Look OAM delivering the 2025 Peter Hollingsworth Honoured Lecture in Brisbane.

Over 100 attendees listening to Dr Burt Look OAM.

Toowoomba AGS Tech Talks Evening

Thursday 30 April 2026

The AGS Queensland Chapter held its first event in Toowoomba! The evening lecture series was at the request of Jian James of Engineers Australia's Toowoomba Regional Group committee.

Technical presentations were delivered by eminent industry speakers from various organisations including consultancies, local government, and contractors.

The Queensland Chapter continues to pride itself as a provider for knowledge sharing and promoting technical excellence as well as enabling great networking opportunities.

The event was a huge success with over 70 attendees from all walks of engineering.

Toowoomba Tech Talks evening at the library.

The program started with a brief introduction of the AGS Queensland Chapter by Vincent Blanchet (Immediate Past Chair) and a brief update from Chris Bridges on Engineers Australia activities, which was followed by technical presentations including:

- Cressbrook Dam Safety Improvement Project – Lessons and Learnings to Date presented by Courtney Shadbolt (Toowoomba Regional Council), Janus Basson (SMEC) and Reid Baldry (Seymour Whyte).
- Managing Expansive Soils in Bulk Earthworks: Challenges and Practical Solutions Applied to a Local Project in Toowoomba by Jaxon Taylor (Engeo).
- Slope Engineering and Risks by Dr Chris Bridges (SLR).
- Toowoomba Range Rail Corridor – Slope Risk Hazards and Maintenance Strategies by Andy Law (SMEC) and Bruce Cheesman (SMEC).

These Technical Events are supported by our Annual Sponsors including Wagstaff Piling (Platinum) and Gold sponsors IGS, EDG, and Black Insitu.

Speakers, and Organising Committee at AGS Toowoomba Tech Talks 2026.

Speaker dinner, Left to Right: Jian James, Trudy Wallington, Andy Law, Bruce Cheesman, Courtney Shadbolt, Jared Priddle, Jaxon Taylor, Vincent Blanchet.

SOUTH AUSTRALIA AND NORTHERN TERRITORY

Chapter Events

The 2026 technical program commenced with the 63rd Rankine Lecture delivered by Professor Kenichi Soga, an internationally recognised leader in geotechnical and infrastructure engineering. Professor Soga drew on his extensive experience across major infrastructure projects and research to discuss how advances in geotechnical engineering were influencing the delivery and longterm performance of complex infrastructure. The presentation emphasised the importance of integrating deep technical understanding with innovation, monitoring, and sound engineering judgement to respond to increasingly complex ground conditions. Members benefited from exposure to global perspectives and challenges relevant to both research and professional practice.

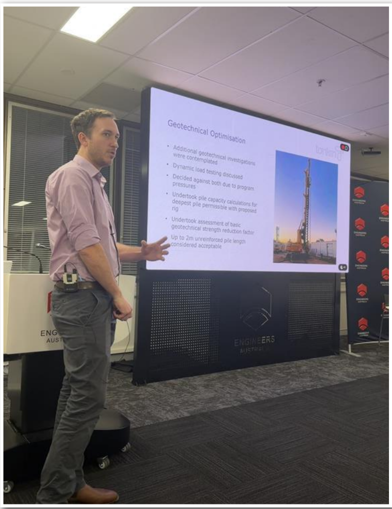
Rankine Lecture – Professor Kenichi Soga

The technical presentation in March focused on brownfield redevelopment, delivered by Scott Williams (Trilogy Consulting) and Helen Boot (Menard Oceania). The speakers examined how historical land use and legacy ground conditions influenced geotechnical risk, investigation strategies, and design outcomes on redevelopment sites. The presentation examined ground improvement techniques and methodologies suited to brownfield conditions, with a focus on selecting appropriate solutions that respond to variable ground conditions and constraints imposed by previous site use. It highlighted the importance of integrating site history, ground improvement strategies, and constructability considerations to manage risk effectively in constrained urban environments. Practical insights were shared on navigating uncertainty and ensuring geotechnical advice remained adaptable as site understanding evolved.

The SA/NT Chapter hosted an International Women's Day Walk and Talk event, providing an informal setting for members to connect outside the traditional technical forum. The event encouraged discussion and reflection on inclusion, professional experiences, and career pathways within the ground engineering community. The relaxed nature of the Walk and Talk allowed for attendees to make many meaningful connections throughout the morning, strengthening the WIAGS community across a range of career stages, and reinforcing the Chapter's commitment to fostering a welcoming and inclusive professional environment.

International Women's Day – Walk and Talk

The April technical presentation was delivered by Jordan NeisBeer of Tonkin Consulting, who examined the importance of flexibility in engineering design through a case study of a commercial development affected by challenging brownfield conditions and historical site use. The presentation described how unresolved geotechnical risks following detailed design prompted a reassessment of foundation solutions, leading to a revised design approach supported by close involvement during construction. Emphasis was placed on calibrating observed ground conditions with design assumptions, managing uncertainty as construction progressed, and maintaining adaptability in decisionmaking. The session highlighted the value of proactive geotechnical engagement in achieving improved outcomes on complex projects.



April 2026 Presentation – Jordan Neis-Beer (Tonkin)

The AGS SA/NT Chapter thanks all presenters and attendees for their contributions and looked forward to continuing a diverse program of technical presentations and networking activities throughout the remainder of the year.

Lauren Amato, SA/NT Chapter Chair

WESTERN AUSTRALIA

There has been some technical seminars and a Rankine Lecture in WA in the first few months of 2026.

Technical seminars

The technical seminars for the year kicked off in March, featuring Dr David Oliveira from Aurecon, who presented on 'Design of High-Performance Rock Bolted Thin Shotcrete Linings: From Basics to Advanced Concepts'. The talk explored how tunnels in poor to fair-quality rock can be stabilised using rock bolts and thin shotcrete linings, which work together to reinforce the rock mass, control localised failures and enable safe stress redistribution during excavation. Advanced design concepts were also covered, including fibre reinforcement, compressive membrane action and non-linear concrete behaviour, with a focus on optimising support performance under dynamic loading conditions.

The seminar was held in collaboration with the Australian Tunnelling Society (ATS) and the WA Ground Control Group (WAGCG), drawing a diverse audience of geotechnical engineers, engineering geologists, structural engineers, technicians, project coordinators and concrete technologists.

Dr David Oliveira sharing a technical seminar on high performance rock bolted thin shotcrete linings

In April, Danny David from Ventia presented on a complex rehabilitation project in a talk titled 'Ranger Uranium Mine Rehabilitation Project: TSF Dewatering for Closure, Pit 3 Wick Drain Installation & Injection Well Drilling'. The talk offered valuable insights into the challenges, innovation and teamwork required to install prefabricated vertical drains (PVDs) over water into underlying tailings, to facilitate drainage and accelerate consolidation to enable remediation. The project navigated a demanding range of challenges, including working in a contaminated marine environment and extreme weather conditions, all within the remote setting of Kakadu National Park in the Northern Territory.

David Danny sharing project challenges and solutions on the Ranger Uranium Mine Rehabilitation Project

63rd Rankine Lecture

On 24 February 2026, Perth hosted the prestigious 63rd Rankine Lecture delivered by Professor Kenichi Soga. Kenichi delivered a thorough and engaging lecture exploring how distributed sensing and data analytics are transforming geotechnical engineering, shifting the field from traditional monitoring approaches toward adaptive, performance-based design and infrastructure management.

Prof. Kenichi Soga (4th from left) in Perth following the Rankine Lecture with AGS WA members

Australian Geomechanics Society Prize, Curtin University

Each year, the AGS supports the AGS Prize at Curtin University, awarded to the recipient of the 'Best Geotechnical Research Project Award'. The prize recognises the most impactful final year project undertaken by final year undergraduate students specialising in geotechnical engineering at Curtin University. It aims to inspire undergraduate students with an interest in geotechnical engineering by recognising their work in research and projects for the betterment of geotechnical engineering knowledge and best practices.

The recipient of the AGS Prize, Curtin University this year was Harikleia Kontorinis. We extend our congratulations to Harikleia for her impressive achievement. The ceremony was held at Tim Winton Lecture Theatre hosted by Curtin University on 10 April 2026.

Donovan Lium (left) on behalf of the AGS presenting the AGS Prize, Curtin University to Harikleia Kontorinis (right).

Eddy Yong, AGS WA Chapter Chair

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The Australian Geomechanics Society was founded in 1970. Its origins lie in the National Committee of Soil Mechanics of the Institution of Engineers, Australia established in 1953 and the call for a corresponding society in rock mechanics. In 1973 the society was expanded to include the third discipline of engineering geology and has remained substantially unchanged since that date.

The society is affiliated with:

- International Association of Engineering Geology and the Environment
- International Society for Soil Mechanics and Geotechnical Engineering
- International Society for Rock Mechanics

The AGS produces *Australian Geomechanics* the newsletter and journal of the Society and specialty conferences, symposia, seminars and workshops, including the four-yearly ANZ Geomechanics conference. *Australian Geomechanics* is published four times a year. The AGS is jointly sponsored by the Institution of Engineers Australia and the Australasian Institute of Mining and Metallurgy.

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7-9	5th Workshop of IGS TC-Reinforcement & ISSMGE TC-218, Mediterranea University of Reggio Calabria, Italy *https://www.geosyntheticssociety.org/events/5th-workshop-of-igs-tc-reinforcement-issmge-tc-218/*
7-9	Pile Driving Contractors Association (PDCA) 26th Annual Conference & Expo - Pile Driving Around the World, Lake Buena Vista, Florida, USA *https://www.piledrivers.org/conferences-and-events/pdca-2026-annual-conference/*
10	Second ISRM Commission Conference on Estimation of Rock Mass Strength and Deformability, Colombo, Sri Lanka *https://www.slrmes.org/*
13-17	13th U.S. National Conference on Earthquake Engineering (13NCEE), Portland, Oregon, USA *https://13ncee.eeri.org/*
21-24	Australian Conference on Rock Mechanics (ACRM), Melbourne, Australia **AGS EVENT** *https://australiangeomechanics.org/meetings/acrm2026/*
AUGUST 2026	
4-6	ANCOLD 2026 Dam Operators Forum, Perth, Western Australia, Australia *https://ancold.org.au/*
4-7	12th International Conference on Short and Medium Span Bridges, Vancouver, British Columbia, Canada *https://www.smsb2026.ca/*
6-10	12th International Symposium on Field Monitoring in Geomechanics 2026, Indore, India *https://isfmg2026.com/*
11-13	Caving 2026, Ulaanbaatar, Mongolia *https://www.acgcaving.com/*
14-16	10th International Conference on Geoscience Education (GeoSciEd X), Adelaide, Australia *https://eventstudio.eventsair.com/geoscoed-conference-2026/*
24-25	4th International Conference on Geotechnical Engineering - Resilient Geotechnics for a Sustainable Future, Colombo, Sri Lanka *https://icgecolombo2026.org/*
24-26	International Conference on Advances and Innovations in Soft Soil Engineering, Delft, Netherlands *https://www.issmge.org/events/international-conference-on-advances-and-innovations-in-soft-soil-engineering-2026*
26-28	Fourth Workshop on the Future of Machine Learning in Geotechnics (4FOMLIG), Seoul, Republic of Korea. *https://fomlig2026.com/*
26-28	X Latin American Congress on Rock Mechanics - an ISRM Regional Symposium, Brasilia, Brazil *https://isrm.net/conference/show/6388*
SEPTEMBER 2026	
1-3	ANCOLD 2026 Tailings Dam Operators Forum, Devonport, Tasmania, Australia *https://ancold.org.au/*
7-8	3rd International Conference on Construction Resources for Environmentally Sustainable Technologies (CREST 2026), Cambridge, UK *https://engage-events.ifm.eng.cam.ac.uk/IC-CREST2026#/*
12-16	GeoQuébec 2026 - 79th Canadian Geotechnical Conference, Québec, Canada *https://geoquebec2026.ca/en/*
13-17	13 ICG – 13th International Conference on Geosynthetics - "Legacy, Evolution & Revolution in Geosynthetics", Montreal, Canada *https://www.13icg-montreal.org/*
13-19	AEG's 69th Annual Meeting - Moving Environmental and Engineering Geology Forward, Chattanooga, Tennessee, USA *https://www.aegannualmeeting.org/*
14-15	1st Scientific Colloquium - Large Scale Testing, Karlsruhe, Germany *https://www.ibf.kit.edu/908.php*
15-19	Eurock 2026 - Risk Management in Rock Engineering - an ISRM Regional Symposium, Skopje, North Macedonia *https://isrm.net/conference/show/6376*
16-18	Fourth International Symposium on Geotechnical Engineering for the Preservation of Monuments and Historic Sites, Athens, Greece *https://tc301-athens.com/*
20-23	The 5th International Conference on Coupled Processes in Fractured Geological Media: Observation, Modeling, and Application, Uppsala, Sweden *https://www.coufrac2026.com/*
20-24	Near Surface Geoscience Conference & Exhibition 2026, Thessaloniki, Greece *Home - EAGE NearSurface*
20-26	Damsweek 20026, Belém, Brazil *https://cbdb.org.br/evento/dams-week-2026*
21-23	2nd International Conference on In Situ Measurement of Soil Properties and Case Histories (INSITU 2026), Bali, Indonesia *https://www.insitu2026.com/*
27-30	6th SEGJ International Symposium - New Frontiers in Geophysics: From Resources to Infrastructure, Sapporo, Japan *https://sites.google.com/segj.org/is-16th/home*

CONFERENCE CALENDAR

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13-16	6th International Conference on Information Technology in Geo-Engineering, Graz, Austria *https://www.icitg2026.com/*
14-16	Hydro 2026 - Adapting to change - Embracing opportunities, Bologna, Italy *https://www.hydropower-dams.com/hydro-2026/*
26-28	2026 ANCOLD Conference - Resilient Dams, Smart Futures, Hobart, Tasmania, Australia *https://ancoldconference.com.au/*
30-6 Nov	XV IAEG World Congress, Delft, The Netherlands *https://iaeg.info/event/xv-iaeg-world-congress/*
NOVEMBER 2026	
4-6	International Conference on Performance-Based Design in Earthquake Geotechnical Engineering (PBD), Puerto Varas, Chile *https://www.pbd-v-chile.com/*
26-27	6th International Conference on Geotechnics for Sustainable Infrastructure Development, Hanoi, Vietnam *https://geotechn.vn/*
MARCH 2027	
17-19	7th International Conference on Grouting and Deep Mixing, Florence, Italy *https://dfi.org/events/upcoming-events/*
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12-14	International Symposium on Ground Improvement (IS-GI Lyon 2027,TC-211 Symposium), Lyon, France *https://www.menard-group.com/isgi-lyon2027/*
23-29	World Tunnel Congress (WTC 2027), Antwerp, Belgium *https://about.ita-aites.org/future-events*
MAY 2027	
11-13	5th International Conference on Shaft Design and Construction (SDC2027), London, UK *https://www.iom3.org/events-awards/5th-international-shaft-design-construction.html*
12-14	International Symposium Cone Penetration Testing CPT '27, Vancouver, British Columbia, Canada *https://www.cpt27.org/*
AUGUST 2027	
31-3 Sep	International Conference on Scour and Erosion (ICSE13), Porto, Portugal *https://icse13.org/*
SEPTEMBER 2027	
21-24	11th European Conference on Numerical Methods in Geotechnical Engineering (NUMGE 2027), Graz, Austria *https://www.tugraz.at/events/numge2027/*
OCTOBER 2027	
17-23	16th ISRM International Congress on Rock Mechanics, Seoul, Korea *http://isrm2027.website.or.kr*
NOVEMBER 2027	
1-4	10th International Congress on Environmental Geotechnics (10ICEG), Kyoto, Japan *https://10iceg.org/*
JANUARY 2028	
9-12	12th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground (IS-Doha 2028), Doha, Qatar *https://www.issmge.org/news/is-doha-2028-early-announcement*
MARCH 2028	
26-29	18th Panamerican Conference on Soil Mechanics and Geotechnical Engineering and Geo-Congress 2028, Chicago, Illinois, USA *https://www.geocongress.org/*
JUNE 2028	
25-30	Eurock2028 - Advances in rock mechanics and rock engineering to cope with increasingly extreme conditions, Aix-en-Provence, France *https://isrm.net/conference/show/6396*
AUGUST 2028	
12-28	38th International Geological Congress - Geosciences for Humanity, Calgary, Canada *https://www.igc2028canada.org/*
SEPTEMBER 2029	
1-5	6th International Conference on Transportation Geotechnics, Southampton, United Kingdom *https://inconference.eventsair.com/cmspreview/ictg-2029*

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AUSTRALIAN GEOMECHANICS SOCIETY

The Australian Geomechanics Society invites PhD students to submit an abstract of their thesis completed in 2026 to *Australian Geomechanics*.

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The thesis should have been completed and accepted within one year of the abstract being published in *Australian Geomechanics*.

The invitation is open to all theses related to geomechanics topics. AGS requests promotion of this initiative among PhD students and academic networks.

The abstracts will be published in the March 2027 issue of *Australian Geomechanics*.

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REBUILDING CONFIDENCE IN CONSTRUCTION MATERIALS TESTING: A PATHWAY TO INTEGRITY, INNOVATION, AND INDUSTRY RENEWAL

Chris Bloxsom
Butler Partners

https://doi.org/10.56295/AGJ6121

1 INTRODUCTION

After more than four decades working across the Construction Materials Testing (CMT) sector as an employee, business owner, NATA Technical Assessor, and long-standing participant in industry reform efforts, I have witnessed the extraordinary value that high-quality testing can bring to Australia's built environment. Construction Materials Testing (CMT) laboratories and technicians, play a foundational role in ensuring that infrastructure performs safely and reliably over its design life. When testing is done well and with integrity, it is invisible; when it is compromised, the consequences can be profound.

In recent years, I and many others with significant experience in the CMT industry have become increasingly concerned about systemic pressures that threaten the integrity of CMT practices across the country. These pressures are not necessarily the result of individual failings or isolated incidents. They can stem from structural issues; particularly the persistent downward pressure on the price paid for testing which can create an environment where corner-cutting becomes normalised, data integrity is compromised, the inherent conflict of interest where testing is commissioned by the earthworks contractor rather than the asset owner, and the credibility of our profession is placed at risk by factors which are difficult for us to influence without a concerted industry-wide effort.

The challenges we face as an industry are not insurmountable and we should recognise that they present an opportunity for the CMT industry to evolve, modernise, and strengthen its role within the broader geotechnical and construction ecosystem. By embracing technology, rethinking responsibility structures, and fostering a culture that prioritises quality over cost, we can build a future where testing integrity is assured, professional standards are elevated, and the industry's reputation is enhanced.

This opinion piece explores the nature of the integrity challenges facing the CMT sector, hopefully illustrates their real-world implications through generalised scenarios, and proposes practical, achievable pathways for reform. My intention is to not just criticise the industry but to encourage constructive dialogue and collective action. The solutions are within reach and generally well understood if we choose to pursue them together.

2 UNDERSTANDING THE INTEGRITY CHALLENGE

2.1 THE PRICE–INTEGRITY PARADOX

It is my view and one shared by many others I talk with, that the core issue confronting the CMT industry is the paradox created by competitive tendering. The clients of CMT laboratories are often under their own budgetary and schedule pressures and naturally seek the lowest possible price for testing services. CMT laboratories, in turn, compete aggressively to win work, frequently reducing margins to unsustainable levels. It may in fact not be immediately evident to the CMT laboratory that the impact of those reduced margins has such an impact on the financial health of their business until profitability declines to such a degree that it can't be ignored. The result is a business environment where the following becomes the case:

- Technicians are expected to complete more tests in less time
- Supervisory resources are stretched thin and technical control is limited or non-existent
- Training budgets are reduced, deferred or mis-directed
- Equipment maintenance and calibration is deferred
- Data verification and checking processes are rushed and often ineffective

- Ethical decision-making becomes strained

This is not necessarily a matter of bad actors, but the inevitable outcome of structural incentives where the market rewards low price rather than high quality and integrity. Those laboratories are pushed toward practices which prioritise throughput over accuracy. Over time, this erodes the integrity of testing and undermines confidence in the results that underpin critical engineering and asset performance decisions.

2.2 THE HUMAN FACTOR

It is no surprise to anyone that CMT technicians are the backbone of the CMT industry. They often work long hours, in sometimes challenging conditions, and carry significant responsibility for the safety and performance of infrastructure. Regrettably many are placed in situations where they must choose between meeting unrealistic deadlines and adhering to rigorous testing protocols.

The significant majority of technicians want to do the right thing and take pride in their work, however when the system rewards speed over accuracy, even the most conscientious individuals can feel pressured to compromise. This has a cultural impact at a laboratory level when the testing accuracy, compliance with the relevant test methods and good practice receives a lower priority than producing reports to a deadline achieved only by corner-cutting and worse.

2.3 THE SYSTEMIC NATURE OF THE PROBLEM

It is vitally important to emphasise that integrity challenges are not confined to any particular region, company, or project type. They arise wherever:

- Procurement of CMT services is based primarily on lowest price
- Testing schedules are compressed to meet construction timelines
- Oversight mechanisms are weak or inconsistent
- Digital traceability is limited
- Accountability for compliance is unclear

These are systemic issues which require systemic solutions and I sense that there is a mood in the industry to recognise the need for change and to translate that recognition into coordinated action. It is worth stating that even the most well managed, profitable and strong quality culture laboratory can become the victim of a “rogue” technician who for reasons known only to himself/herself will cut corners or otherwise compromise the quality and veracity of test data obtained. As discussed later, the proposed technology-driven solutions will minimise or see an end to this type of behaviour. With some current Laboratory Information Management Systems (LIMS) in use, this process is arguably underway to various degrees.

3 WHAT HAPPENS WHEN INTEGRITY FAILS

To understand the real-world implications of compromised testing integrity, it is helpful to consider two generalised scenarios. These examples are not tied to specific projects or organisations; rather, they reflect patterns described in a number of reported articles over a long period of time, and discussions held with senior industry figures from time to time. What is not known nor discussed below are the possible ramifications suffered by the CMT laboratories where these scenarios have been uncovered, including commercial and criminal litigation against organisations and individuals.

3.1 SCENARIO 1: COMPACTION TEST MANIPULATION ON A MAJOR INFRASTRUCTURE PROJECT

On a large infrastructure project, soil compaction testing was being conducted under an intense schedule pressure. The construction program allowed little time for rework, and the client expected a rapid turnaround of test results. The laboratory engaged for the project had bid aggressively to secure the contract, leaving limited resources for supervision and quality control.

Independent auditors later discovered that a significant proportion of compaction test results had been systematically falsified. In some cases, technicians had adjusted moisture content values to achieve passing results. In others, density readings were altered to avoid reporting non-compliance. The manipulation was not the work of a single individual; it reflected a broader culture shaped by unrealistic expectations and inadequate oversight.

The consequences were substantial. Portions of the project had to be excavated and reworked, leading to delays and increased costs. More importantly, confidence in the testing process was shaken. Engineers, contractors, and regulators were forced to question whether other results could be trusted. This leads to an understandable crisis of credibility for the CMT industry where due to the actions described above, all testing becomes suspect and devalued in the eyes of the those who need to rely on accurate and reliable compliance services.

This scenario illustrates how integrity failures can emerge not necessarily from malicious intent but from systemic pressures that make non-compliance seem like the only viable option.

3.2 SCENARIO 2: FORGED CONCRETE STRENGTH CERTIFICATES

In another case, concrete strength certificates were found to have been forged. The laboratory responsible for testing was operating under severe financial pressure, having secured the contract through a lowest-price tender. Technicians were overworked, equipment calibration was overdue, and management oversight was limited.

When discrepancies were identified during a routine review, further investigation revealed that several certificates had been fabricated without corresponding test data. The concrete in question did not meet the specified strength requirements, resulting in structural deficiencies that required costly remediation.

Again, the issue was not isolated and reflected a broader pattern in which laboratories, squeezed by price competition, cut corners to maintain profitability. The long-term cost to the project far exceeded any short-term savings achieved through low-cost procurement.

These scenarios underscore the importance of addressing integrity challenges proactively. They also highlight the opportunity for the industry to adopt new approaches that reduce the likelihood of such failures occurring in the future.

4 SOLUTIONS FOR BUILDING INTEGRITY

The integrity challenges facing the CMT industry are serious, but they are not insurmountable. By embracing innovation, rethinking responsibility structures, and fostering a culture of quality, we have it within our ability to build a stronger, more resilient industry. The following sections outline and discuss potential, practical, and achievable solutions across three key domains. At this stage of my career, I would best leave the “nuts and bolts” of these solutions to those who are much more integrated at a technical proficiency level of the systems which can drive and manage the solutions the industry needs.

5 TECHNOLOGY-DRIVEN SOLUTIONS

In my extensive and ongoing discussions with industry colleagues and those in my business, I understand that a range of technology offers some of the most promising pathways for improving testing integrity. Many of the tools needed already exist and the challenge is to integrate them effectively into industry practice.

5.1 AUTOMATED DATA ACQUISITION SYSTEMS

Automated data acquisition systems reduce the reliance on manual data entry, which is one of the most common sources of error and manipulation. By capturing data directly from testing equipment, these systems:

- Improve accuracy
- Reduce opportunities for tampering
- Increase efficiency
- Provide consistent, standardised outputs

For example, automated compaction testing equipment can record density and moisture content values directly, eliminating the need for technicians to transcribe results. Similarly, automated concrete compression machines can capture load and displacement data in real time, ensuring that results are recorded accurately and transparently.

5.2 CLOUD-BASED REPORTING AND TAMPER-EVIDENT RECORDS

Cloud-based reporting platforms allow laboratories to store test data securely, with built-in audit trails that make any alterations visible. These systems can:

- Provide real-time access to results for clients and regulators
- Reduce the risk of data loss
- Ensure that all changes are logged and traceable
- Support remote auditing and oversight

Tamper-evident digital records are particularly valuable in environments where multiple stakeholders rely on the same data. They create a shared source of truth that enhances transparency and accountability.

5.3 AI AND MACHINE LEARNING FOR ANOMALY DETECTION

Artificial intelligence and machine learning algorithms can analyse large datasets to identify patterns and anomalies that may indicate non-compliance. For example, AI systems can flag:

- Repeated identical values in compaction tests
- Unusual distributions of concrete strength results
- Inconsistencies between field and laboratory data
- Deviations from expected statistical patterns

These tools do not replace human judgement; they enhance it. By providing early warnings of potential issues, AI systems allow supervisors and auditors to focus their attention where it is most needed.

5.4 DIGITAL TRACEABILITY SYSTEMS

Digital traceability systems—such as QR-coded sample tracking, GPS-enabled field devices, and blockchain-based recordkeeping—can provide end-to-end visibility of the testing process. They ensure that:

- Samples are correctly identified
- Chain-of-custody is maintained
- Field and laboratory data are linked
- Records cannot be altered without detection

These systems build trust by demonstrating that testing processes are transparent, secure, and verifiable.

6 STRUCTURAL AND RESPONSIBILITY-BASED SOLUTIONS

As an industry, it is understood that technology alone cannot solve integrity challenges and that structural reforms are needed to clarify responsibility, strengthen accountability, and ensure that testing is valued appropriately within the construction process. This “valuation” is something which needs to be embraced by clients of CMT laboratories in seeing that the testing has an intrinsic value to the project and not just a box to be ticked when completing the QA report. It is something that the CMT industry has brought upon itself when our clients place low value on the accuracy and integrity of testing as a result of previous experience of compromised test reporting or similar undesirable outcomes.

6.1 RECONSIDERING ASSET OWNER RESPONSIBILITY

One of the most significant opportunities for reform lies in rethinking who is responsible for testing and compliance. Currently, testing is often procured by contractors, who may prioritise cost and speed over independence and accuracy.

A more robust approach would be for asset owners, whether public or private, to take direct responsibility for engaging testing laboratories. This would minimise the inherent conflict of interest for the CMT laboratory which is working for a contractor who is motivated more by cost and speed and is potentially willing to compromise on quality.

In the early stages of my career in the CMT industry, the majority of testing undertaken for projects fell under the direction of the Principal (asset owner generally), his representative such as the Project Superintendent in the contractual nomenclature of that time. The contractor would “present” a portion of the works for testing and inspection by the Superintendent who would direct the testing laboratory to perform tests and report as required by the project specifications. In some instances, a contractor may have engaged a CMT laboratory to undertake “control” testing to assist them in measuring the quality of construction materials and work practices such as compaction control, etc., which is a valuable addition to construction practice when it is done well and with integrity.

Should the asset owner again take direct responsibility for engaging CMT laboratories, this would:

- Reduce conflicts of interest
- Ensure that testing is aligned with long-term asset performance
- Encourage laboratories to prioritise quality over price
- Strengthen accountability for compliance

When asset owners control testing procurement, they are more likely to value integrity, reliability, and long-term performance.

6.2 RECOGNISING THAT NON-COMPLIANCE RISK ULTIMATELY SITS WITH ASSET OWNERS

Regardless of who procures testing, the ultimate risk of non-compliance rests with the asset owner. If testing is compromised, it is the owner who bears the cost of remediation, reputational damage, and potential safety consequences. Even where the responsibility can be shifted to the contractor for recovery it will take significant time and cost resources to effect this outcome.

Recognising this reality can drive more informed procurement decisions. Asset owners who understand the true cost of non-compliance are more likely to invest in high-quality testing services, robust oversight mechanisms, and long-term partnerships with reputable laboratories.

6.3 STRENGTHENING OVERSIGHT AND GOVERNANCE

Any move toward structural reforms should also include enhanced oversight mechanisms such as:

- Independent third-party audits
- Regular NATA assessments
- Clear escalation pathways for reporting concerns
- Transparent performance metrics for laboratories

These measures will substantially lead to the creation of an environment where integrity is expected, supported, and verified.

7 REGULATORY AND CULTURAL SOLUTIONS

Regulation and culture are closely intertwined. Effective regulatory frameworks support ethical behaviour, while strong industry culture reinforces compliance. Together, they create the conditions for sustainable integrity.

7.1 MOVING BEYOND LOWEST-PRICE PROCUREMENT

The shift from lowest-price procurement to value-based procurement is essential. This does not mean abandoning cost considerations; it means recognising that quality, reliability, and integrity are equally important.

Value-based procurement can include:

- Weighted evaluation criteria
- Minimum quality thresholds
- Past performance assessments
- Requirements for digital traceability
- Incentives for innovation and continuous improvement

These approaches are already well known, but not always well implemented, and they reward laboratories which invest in quality and necessarily discourage those which rely on unsustainable pricing strategies.

7.2 RIGOROUS THIRD-PARTY AUDITS

Independent audits provide an objective assessment of laboratory performance. They can identify:

- Gaps in training
- Inconsistencies in data
- Equipment calibration issues
- Process deviations
- Cultural or ethical concerns

Regular audits, combined with transparent reporting, help build trust and ensure that laboratories maintain high standards. In general, any CMT laboratory undertaking works on State Road Authority projects (my experience is Queensland-based); Department of Transport & Main Roads (TMR) can be and often is subject to surveillance audits by very experienced and practical technicians from TMR. It may be incumbent on the CMT industry to work with NATA and others to develop a more universal third-party audit and surveillance authority to enhance confidence in the CMT industry.

7.3 MANDATORY DIGITAL TRACEABILITY

Digital traceability should be considered to become a standard requirement across the industry. It enhances transparency, reduces opportunities for manipulation, and provides a clear record of compliance.

Regulators and industry bodies such as NATA and the likes of TMR can support this shift by:

- Updating standards to require digital recordkeeping
- Providing guidance on best-practice systems
- Encouraging interoperability between platforms

7.4 UPDATED STANDARDS AND INDEPENDENT VERIFICATION

Australian Standards and industry guidelines play a critical role in shaping testing practices. Updating these documents to reflect modern technologies and integrity expectations can drive significant improvements.

Independent verification—whether through third-party laboratories, cross-checking, or digital validation—adds an additional layer of assurance.

7.5 EDUCATION, TRAINING, AND ETHICAL CULTURE

Ultimately, I believe we all understand that integrity is a cultural issue. Technology and regulation can support ethical behaviour but cannot replace it. As they say, “culture eats strategy for breakfast” and it is my view that the industry must invest in:

- Technician training
- Supervisor development
- Ethical leadership programs

- Industry-wide education initiatives
- Mentoring and professional pathways

A strong ethical culture empowers technicians to speak up, encourages managers to prioritise quality, and reinforces the industry's commitment to integrity.

7.6 PRIORITISING LONG-TERM REPUTATION OVER SHORT-TERM GAINS

The most successful laboratories are those that recognise the value of long-term reputation. Clients trust them, regulators respect them, and employees are proud to work for them. Prioritising reputation over short-term gains is not only ethical—it is good business.

8 OVERCOMING BARRIERS TO CHANGE

While the solutions outlined above are achievable, they nevertheless will require collective commitment to overcome the barriers to change including:

- Resistance to new technologies
- Concerns about cost
- Lack of awareness
- Fragmented industry structures
- Competing commercial pressures

These barriers can be overcome through:

- Collaborative industry forums
- Pilot programs and case studies
- Shared digital platforms
- Clear regulatory guidance
- Leadership from asset owners and government agencies

Change is most effective when it is driven by a shared purpose and supported by practical tools. I am not underestimating the scale and difficulty of what is needed in the CMT industry and believe that things have to change at our behest and not wait until something less palatable is imposed as a result of a continuing decline in integrity.

9 CONCLUSION: A CALL TO ACTION

The integrity challenges facing the construction materials testing industry are real, but they are not insurmountable. These challenges present an opportunity to modernise our practices, strengthen our culture, and enhance our contribution to Australia's built environment.

By embracing technology, rethinking responsibility structures, and fostering a culture of quality and ethics, we can build an industry that is resilient, respected, and future-ready. The solutions are within reach, but what is needed now is collective commitment.

As professionals, regulators, asset owners, and industry leaders, we have a shared responsibility to ensure that testing integrity is never compromised. The safety, reliability, and performance of our infrastructure depend on it.

The path forward is clear. The opportunity is significant. And the time for action is now.

CRediT authorship contribution statement

Christopher Bloxsom: Writing - original draft.

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RETHINKING EARTHWORKS QUALITY TESTING

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ABSTRACT

Earthworks quality assurance has traditionally relied on density and moisture content testing along with California Bearing Ratio (CBR) as the primary indicators of compaction and performance. While these methods provide a useful and long-standing framework for earthworks construction control, their dominance has created a "density illusion": the belief that achieving a high target density ratio equates to long-term performance reliability. This illusion has constrained the adoption of more representative, performance-based approaches, despite advances in geotechnical science and measurement technologies. The historical perspective of density being the preferred earthworks quality test method is re-examined with comparisons with current (past 20 years) test methods.

Results of surveys show there are many common geotechnical tests that are not trusted. This is due to testing variability, correlations and interpretations of these tests. Each test is treated as a black box, but in reality the tests are laboratory or field models with associated assumptions. High precision is not the same as accuracy.

Statistical techniques such as dendrogram analysis and correlation matrices are used to show patterns and identify relationships between different test variables and compared with results in a correlation matrix. Such comparative analyses demonstrates that reliance on CBR and density testing indices can obscure other critical performance indicators. The high precision of density testing hides its inaccuracy to assess key design parameters. Correlation errors occur when relating to more accurate tests. Density testing is only step 1 of a 3 stage quality assurance process.

1 INTRODUCTION

Common practices in geotechnical engineering seem to have many silent inconsistencies that as an industry we do not question as the test or practice is widely used. This anchoring and availability bias as a cognitive dissonance was discussed in Look (2025a). The historical legacy anchoring and construction material testing (CMT) is first examined with a comparison with our trust with common geotechnical testing. This trust was determined from a poll survey of practicing geotechnical engineers and engineering geologists. The poll showed as an industry we continue to use the results of tests we do not trust. Case studies are then used to examine the reliability of such tests.

Another poll compares what attributes of testing we most desire for quality control equipment. Again, this shows the dichotomy of industry practice versus what we want to see in a practice.

Many tests were developed with a different technology or material type. In an attempt to standardise, early tests then become universal and applied to different materials and have continued to a time when technology has changed. As an industry we then adapt the tests rather that start over, so as not to criticise tests which have served industry well in the past. We are openly afraid of what is unfamiliar, uncertain or requires different skills. This presupposes knowledge is static. It is not. This should not be viewed as not adhering to principles developed to avoid past failures.

As individuals, the advice that serves you at 25 years old does not apply at 50 years old. That is stagnation. Growing up should not mean being a slave to the past. This applies to our profession, as we develop and know more, the methodology developed in our first 25 years past may not now be applicable with different processes or changing technology. That is not suggesting the founding fathers being wrong in the past. This is about growing up as a profession in the present.

This means accepting precision is not the same as accuracy on our tests. In analysis a factor of safety reported to 3 decimal places (say 1.543) implies 3 decimal places of precision (± 0.001). Experienced engineers accept that value as incorrect. Similarly, an undrained strength in a clay with a test value (say 32 kPa) is not correct to 1 kPa. Our founding fathers in geotechnics got around such inaccuracies by "classification". The strength then became a classification range (firm clay with 25 to 50 kPa range). This indexing accounts for errors with any single test value.

Words like "optimum" and "maximum" in a technical application should not be confused with an equivalent translation word of "best". This equivocation has led to many misunderstandings in the CMT industry. Are we confusing the most used tests with the most trusted?

2 SURVEY OF PRACTICING ENGINEERS

Surveys of different groups over the years were carried out (and were not specific to this paper) to establish industry practices. A few of these results are discussed.

2.1. GEOTECHNICAL TESTS TRUSTED AND NO TRUSTED

A word cloud poll survey of geotechnical professionals was carried out in November 2025 at the Queensland Geotechnical symposium for various levels of experience. A similar survey was carried out in New Zealand in October 2025 but without the age grouping. The survey responses on the question of geotechnical tests to be trusted or not trusted is summarised in **Figure 1**. Each grouping represents between 11 to 17 responses. This is a summary only as many other tests with 1 or 2 mentions only are not shown.

There is a clear conflict in the geotechnical industry practice with the following rankings:

1. The SPT is the most trusted test overall. The SPT is also the least trusted test overall.
2. The CPT is the 2nd most trusted test overall. The DCP is the 2nd least trusted test overall. Note the DCP is also trusted by some practicing engineers
3. The Vane Shear test is the 3rd most trusted test. The CBR is the 3rd least trusted test. The pocket penetrometer (PP) was also mentioned as a test not to be trusted

The CPT is trusted, but the survey did not provide any distinction between the many variants of the test e.g. seismic CPT or CPTu.

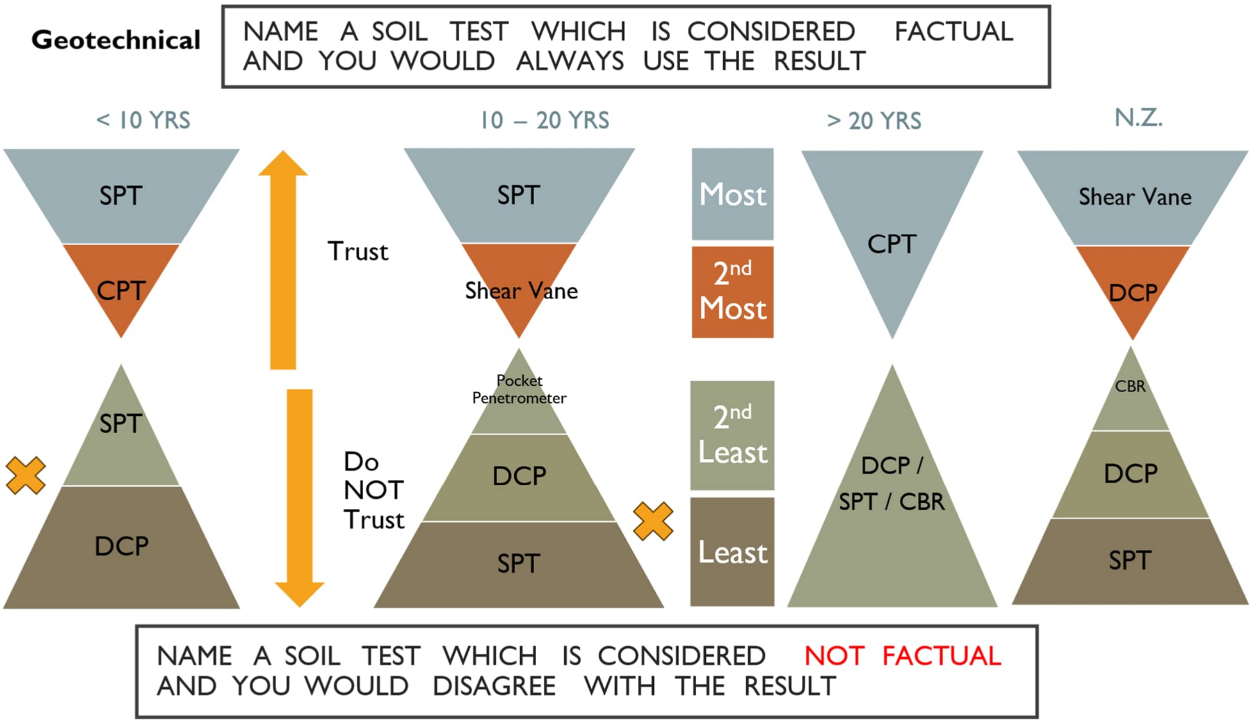


Figure 1: Survey of geotechnical professionals in 2025 on test to be trusted or not trusted. Each grouping represents between 11 to 17 responses with 54 responses in total – a yardstick only

The dichotomy of using a test and trusting its value has been exposed in this survey. One wonders if the respondents may have confused the "most used" ≈ "most trusted". The geotechnical testing for DCP, SPT, CBR and PP are ubiquitous and raises a fundamental philosophical issue on whether we differentiate between high usage and "truth" of a test. The availability bias was discussed in Look (2025a). Our judgement is influenced by readily available information and what springs easily to mind, rather than its "truth". As engineers we chase numerical values as a fact where there are many other factors affecting how that number should be applied. Many tests have underlying assumptions, and the procedural aspects according to standards can be trusted. Directly using the test results is another matter.

2.2. TESTS TRUSTED AND NO TRUSTED BY CIVIL ENGINEERS

When civil engineers and project managers (non-geotechnical specialists) are asked the same question, there are different sets of trusted and non-trusted words (**Figure 2**). Construction control tests such as OMC and Density Ratio tests are not to be trusted by this cohort. In all cases the experience level was approximately 10 years (90% between 5 to 25 years) but the contractor survey respondents were in the 5 - 10 year experience level.

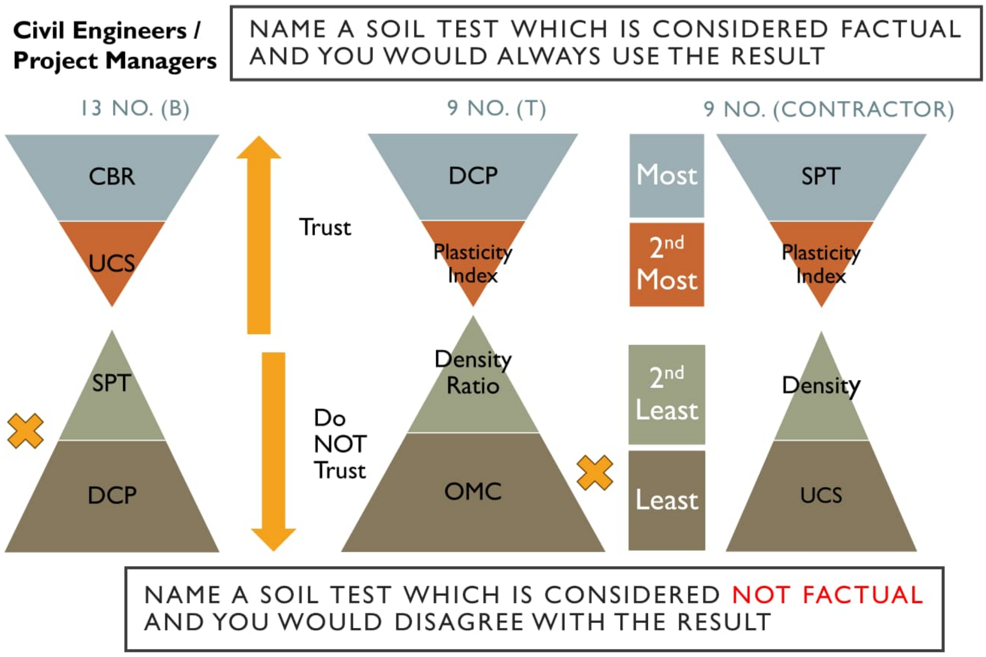


Figure 2: Survey of civil engineers and project managers in 2025 on tests to be trusted or not trusted. Each grouping represents between 9 to 13 responses with the Nos shown (31 total in survey). (B)and (T) represent the survey location

The responses to the question on “At what moisture content does the maximum CBR occur?” show most practicing professional believe this is at the optimum moisture content (OMC) (**Table 1**). Only contractors are divided on this response with 50% also believing the maximum CBR may not be at the OMC. The common fallacy of the maximum CBR being at the OMC will be discussed in subsequent sections, as a key focus of this paper. The CBR test as part of construction material testing (CMT) has many assumptions to be appropriately applied in practice.

Table 1: Responses to multiple choice question: “At what moisture content does the maximum CBR occur?”

Survey Group A, B. T and (No.)	Geotechnical (17 No.)	Civil engineers and project managers			
		A (10 No.)	B (15 No.)	T (15 No.)	Contractor (10 No.)
Maximum CBR @ OMC	59%	63%	93%	93%	50%
Dry of OMC	29%	21%	0%	0%	20%
Wet of OMC	0%	5%	0%	0%	10%
Unknown	12%	11%	7%	7%	20%

Overall, these surveys show practicing and experienced engineers have a highly variable opinion on whether a test is trustworthy.

2.3. DESISABLE ATTRIBUTES IN A TEST EQUIPMENT

A survey of 54 engineers (Look, 2018) on ranking what attributes are desirable in an earthworks quality control test equipment showed that accuracy is the most preferred attribute of any test. The preference ranking order for attributes of a test equipment from that survey was (**Figure 3**):

1. Accuracy
2. Precision
3. Time to do test
4. Time to report test

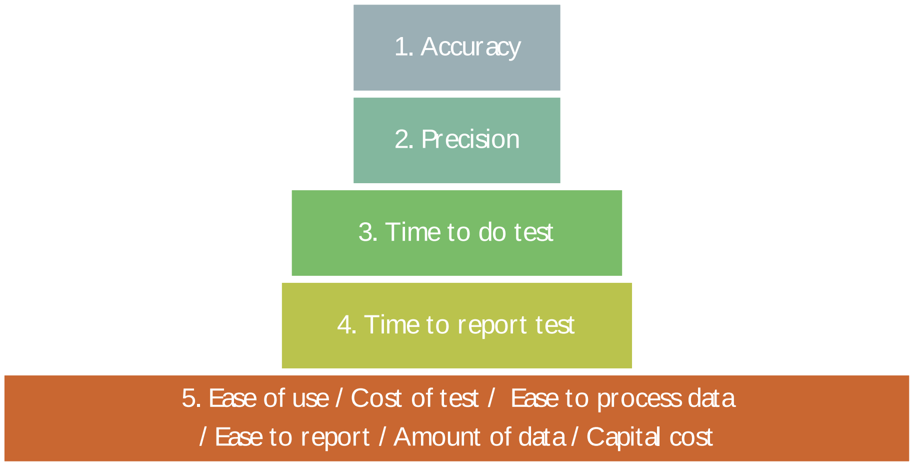


Figure 3: Survey in 2017 on what test attributes are desirable in quality control

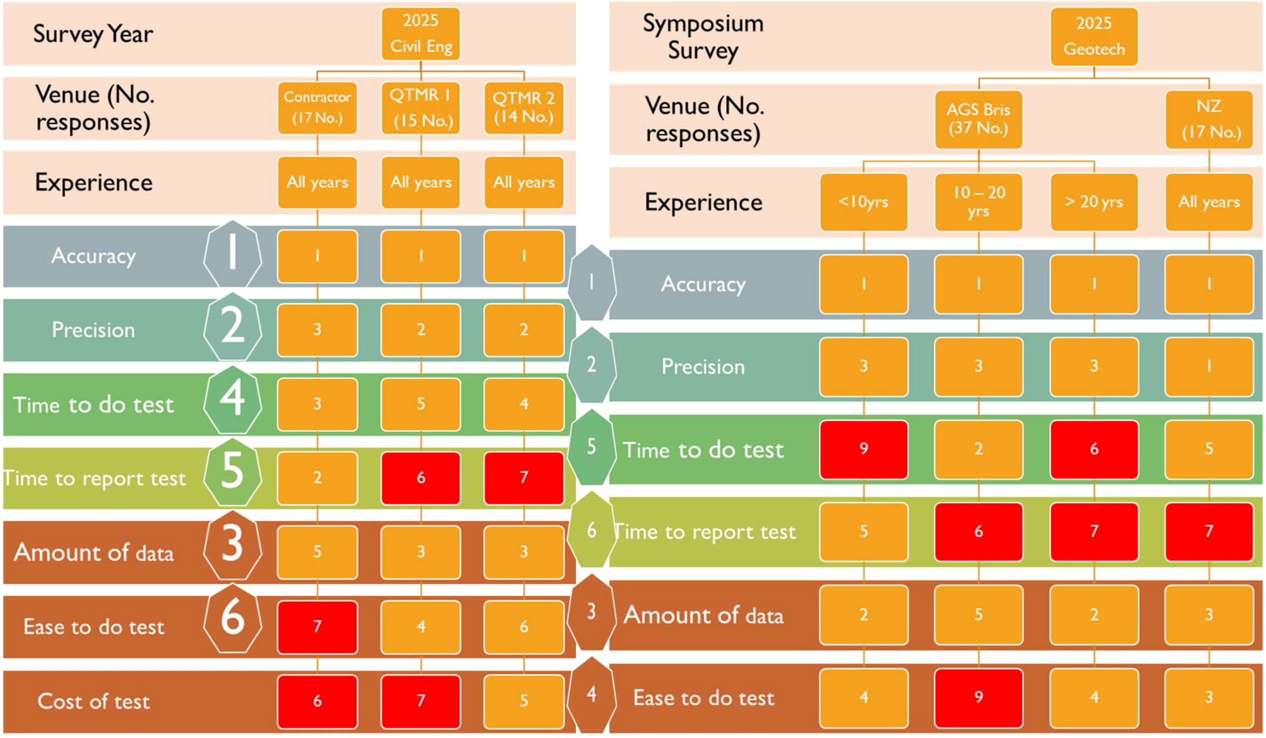
The other factors were closely ranked: Ease of use; Cost of test; Ease to process data; Ease to report; Amount of data obtained; and Capital cost of equipment. That survey was repeated in 2025 in various groups (**Figure 4**) with an attempt to differentiate for:

- Years of experience (< 10 years ; 10 – 20 years ; and > 20 years)
- Industry work (contractor / government (civil Engineers, project managers and inspectors) / geotechnical engineers and engineering geologists
- Location (New Zealand and Queensland)

The survey results shows accuracy and precision are overall ranked number 1 and 2, respectively, in all cases. However, amount of data was now (2025) ranked number 3. Times to do and report test are now (2025) not in the top 4 critical issues for geotechnical professionals with ease of test taking precedence. This order changes for the grouping of contractors, civil engineers and project managers.

An interesting outlier is the time to do the test for those with less than 10 years of experience. They are likely the ones doing the test, and less likely to be concerned with time and cost as their older colleagues. Conversely a contractor ordering the test has this as highly ranked. Industry would consider amount of data as important (No. 3 ranking) while contractors (who are typically project managers ordering the test) consider time to do and report the test as more important.

These factors will now be discussed in light of available construction material testing (CMT) methods. This paper discussion will focus mainly on the anomalies in **Figure 2** and not **Figure 1**, as the former is more associated with CMT while the latter is associated with site investigation overall.



Civil engineers and project managers **(b) Geotechnical engineers and engineering geologists**

Figure 4: Survey in 2025 on what test attributes are desirable in quality control

2.4. TRUSTED RESULTS AND PRECISION

A brief discussion on PP, SPT and DCP is provided below. These tests are not associated with CMT but were shown as conflicted in their reliability. The CPT is generally one of the more trusted field tests. However, there are source of errors associated with calibration and the load range of the cone when used in soft clays (Scholey, 2024).

Sample size often affects the test results. The pocket penetrometer (PP) is a simple test commonly used as a measure of the undrained shear strength. Its simplicity, low cost and availability often tempt many field engineers into its overuse. Due to the small test area, the PP will likely overestimate the strength in a fissured clay. The argument heard is that any test result is better than no test result. Look (2025a) discusses the availability bias as PP test results are easily obtained, but this does not mean they are useful, with often meaningless results that may lead to wrong data for analysis if such test numbers are applied (Look, 2004).

SPT N- values require an energy correction to be used in design (Look, 2025a). SPTs with high and low values are typically subject to a wide interpretation. A flawed circular argument is often used as follows:

- A clay is very soft, therefore N value ≤ 1 is correct. However, the reverse logic does not necessarily apply.
    - If N value ≤ 1 does not mean a clay is very soft.
- A driven pile is unlikely to refuse at N value < 50. Again, the reverse logic leads to an incorrect interpretation
    - If N value > 50, stating a pile has reached refusal is incorrect

At high SPTs, relative density is uncertain and the strength or modulus has a wide interpretation (**Figure 5**). This uncertainty applies when the SPTs are used in residual soils or gravels. A similar uncertainty occurs at high compaction levels when the dry density ratio (DDR) is used to infer strength or modulus. There are similar issues with correlations and material type when DCPs are used, especially at DCPs < 2 or > 10 blows / 100 mm.

The variability of test results would affect whether one would trust the results. The variability can be judged by the coefficient of variation (COV) and is discussed in Look (2024 and 2025b). Below a COV of 35% there would be a general agreement of a “uniform” result or section of a site. Above a COV of 80% the site should be considered non uniform and at COV of 60% the site is unlikely to be uniform.

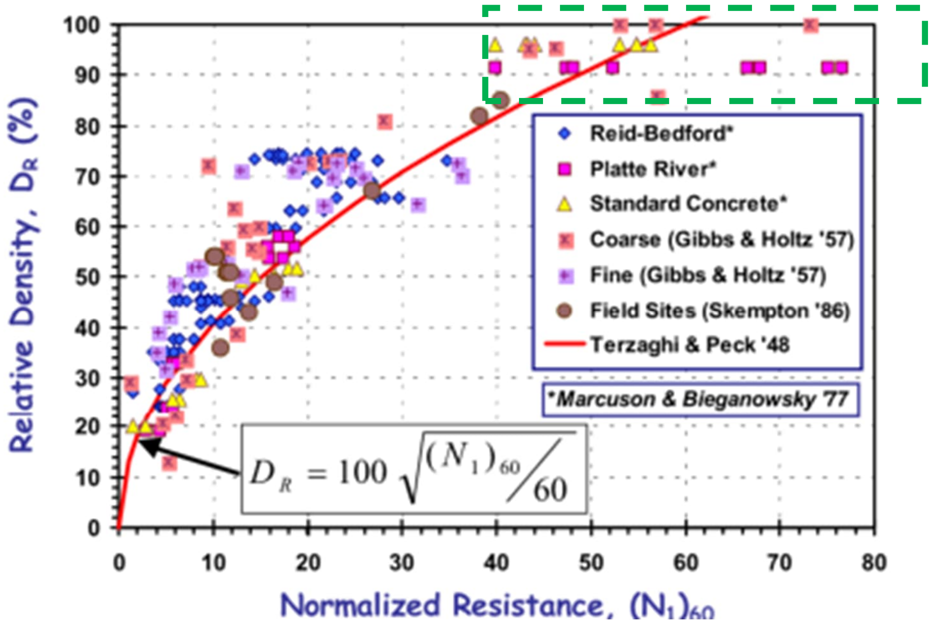


Figure 5: Relative density of clean sands from SPT (Mayne et al., 2002). Highlighted box shows the above D_r of 85% (N_{60} > 40), the corresponding strength or modulus would be subject to a wide interpretation

Look (2024) associates high COV with non-normal probability density functions (PDF) and there would then be 4 opinions for every 5 engineers on the "design" value and outside of a target zone (**Figure 6**). Thus a "uniform" site or selecting design values should use the COV to avoid too wide a variability. The trusted / non trusted test results and their comparative COVs are provided in **Table 2**. The typical COV was obtained from the various references in Look (2014) but based mainly on the seminal work of Phoon and Kulhawy (1999) on inherent testing variability.

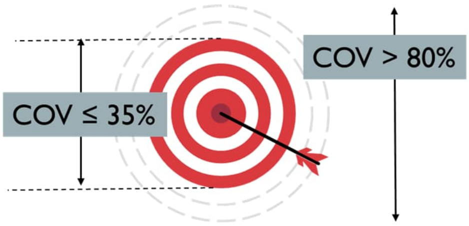


Figure 6: When can normal PDF be applied and a site is uniform vs non uniform

Table 2: Inherent soil test variability and trust

Measurement	Trust	Typical COV (%) without outliers	Comment
Shear Vane CPT Plasticity Index	Yes	25% (Clay) 30% (Clay) ; 40% (Sand) 24%	Correction factor for Plasticity Index required Improves with CPTu or seismic cone. Calibration required Discards material retained on 425 micron sieve
SPT DCP	Yes / No	40% 45%	Energy correction required before using as a design value Increased COV for DCP blows / 100mm < 5
MDD OMC CBR	No No Yes / No	3% 20% 40%	Tests requires curing and removal of over size. Above 20% oversize test is not applicable. Correction for oversize content not more than 20%. Lab compaction energy may not be the same as compaction field energy (varies with equipment used)
UCS	No	23%	The intact UCS value does not apply to apply field value for a jointed rock mass

The comparison of trust versus COV show no clear relationship. One may conclude that experience and site observations are the predominant reasons for trusting a test result. All the tests which industry does not trust have a disconnect between lab and field conditions.

The following 2 case studies from the one site illustrate the COV for a near uniform site. An area of the site with a soaked CBR average value of 8% is shown in **Figure 7** for 15 test results. This material has a COV of 39% which is comparable to the typical COV of 40% (**Table 2**). Selecting a design value is no trivial matter even for a uniform site. A designer may choose the lowest test value of CBR 5%, a lower quartile value of 6% or a median value of 7% depending on the type of project. The exponential is the best fit PDF in this case using statistical goodness of fit tests.

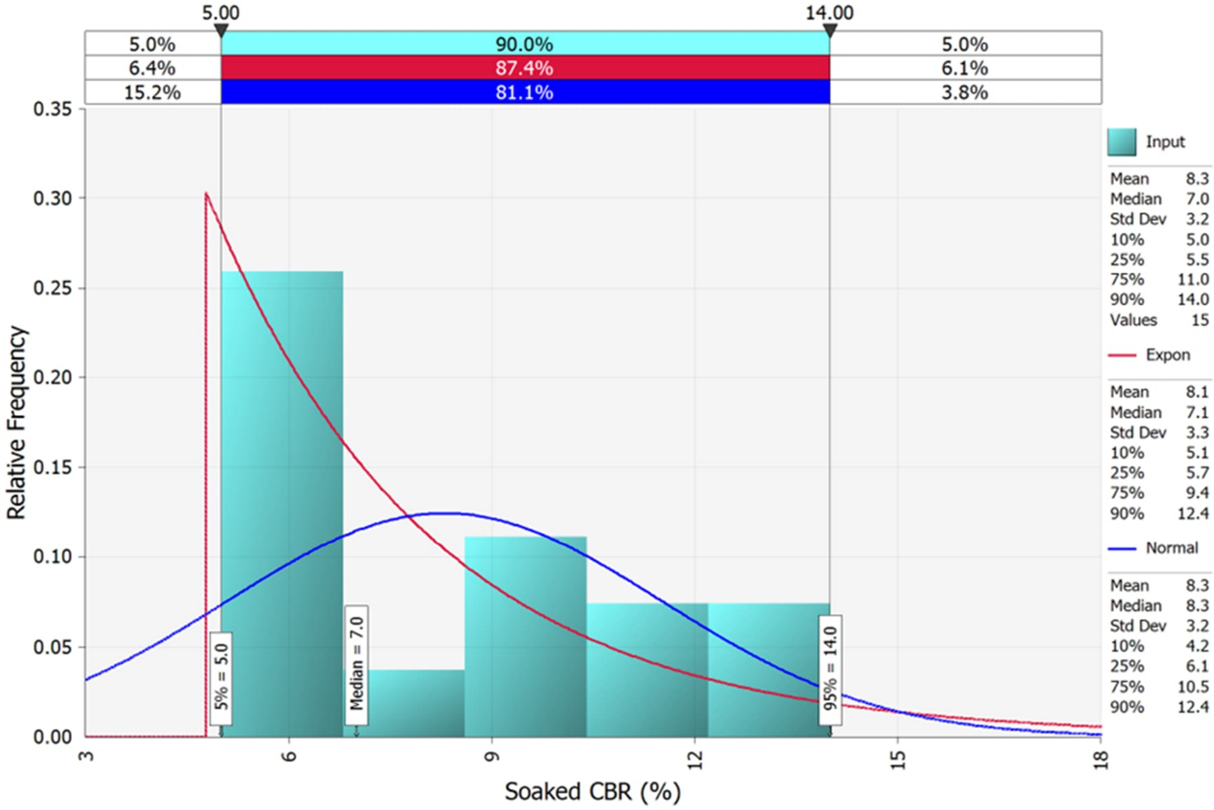


Figure 7: Clayey Sand (SC) material with 34% fines and WPI = 246

An adjacent source material at the same site was classified as Sandy Clay (CL) with a soaked CBR median value of 6% is shown in **Figure 8** for 15 test results. A minor change in fines and weighted plasticity index (WPI) results in a 2% change in the average CBR value. This would be associated with a significant increase in pavement thickness. In this case the normal PDF is the best fit curve and with a COV of 38%. A designer may choose the lowest test value of CBR 1.5%, a lower quartile value of 5% or a median value of 6% depending on the type of project.

3. A SHORT HISTORY OF QUALITY TESTING

The California Bearing Ratio (CBR) test was primarily developed to assess the load-bearing capacity of soils used in highway construction. The CBR is used with the Proctor density test at various compaction levels.

The results of the standard Proctor test should be considered in conjunction with the weight of the roller (and any vibratory enhancement) + lift thickness + number of passes to equate equivalent energy levels. A heavier compaction roller can achieve greater compaction energy and increased soil density when properly operated. Other factors, such as the type of soil, moisture content, compaction method, and roller type, also play significant roles in achieving desired compaction results.

We often default to density as if it is a proxy for performance. At the macro level, strength increases with density e.g. comparing soil with concrete or steel (**Figure 9**). But density alone tells us almost nothing about how a material will behave under load. Strength and stiffness in soil are also dependent on its confinement and stress levels. Appendix A shows how this illustration (**Figure 9**) can be misleading using corelated data.

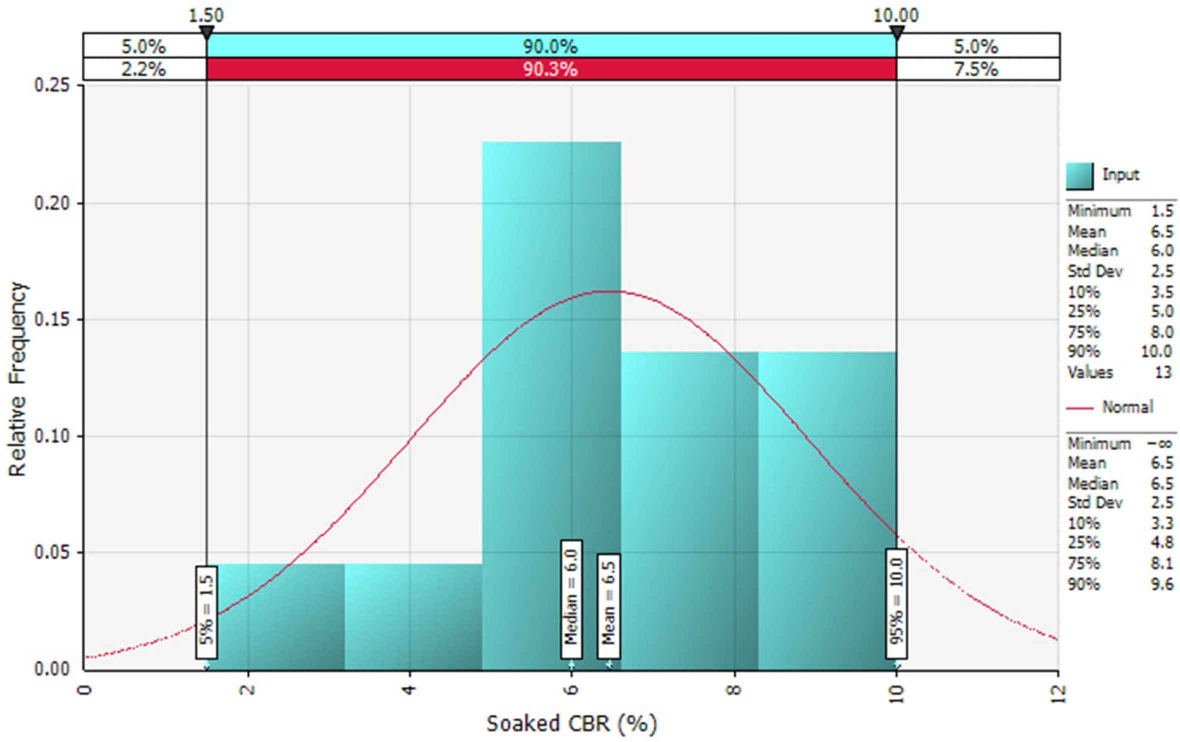


Figure 8: Sandy Clay (CL) material with 38% fines and WPI = 984

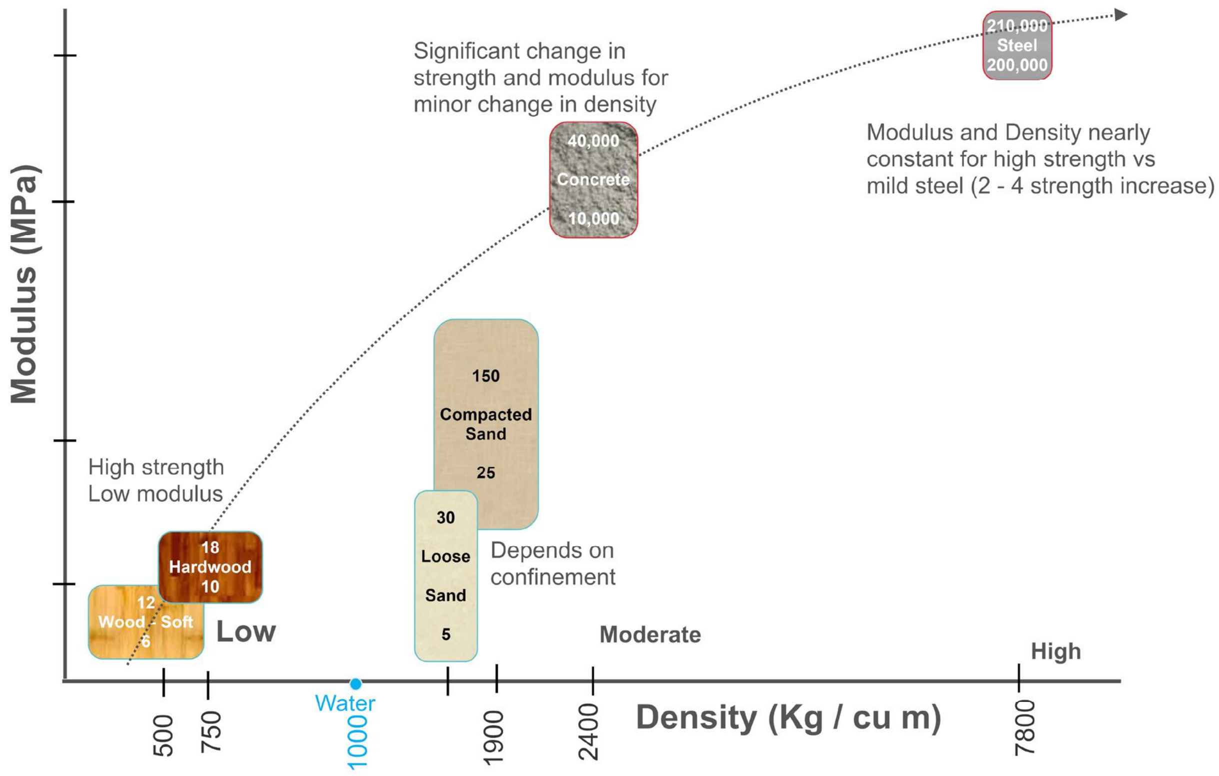


Figure 9: Increase in modulus with density

The maximum dry density (MDD) in soils is dependent on the uniformity coefficient (**Figure 10**). Well graded soils (a high uniformity coefficient) will have a higher MDD than a uniformly graded (low uniformity coefficient) soil.

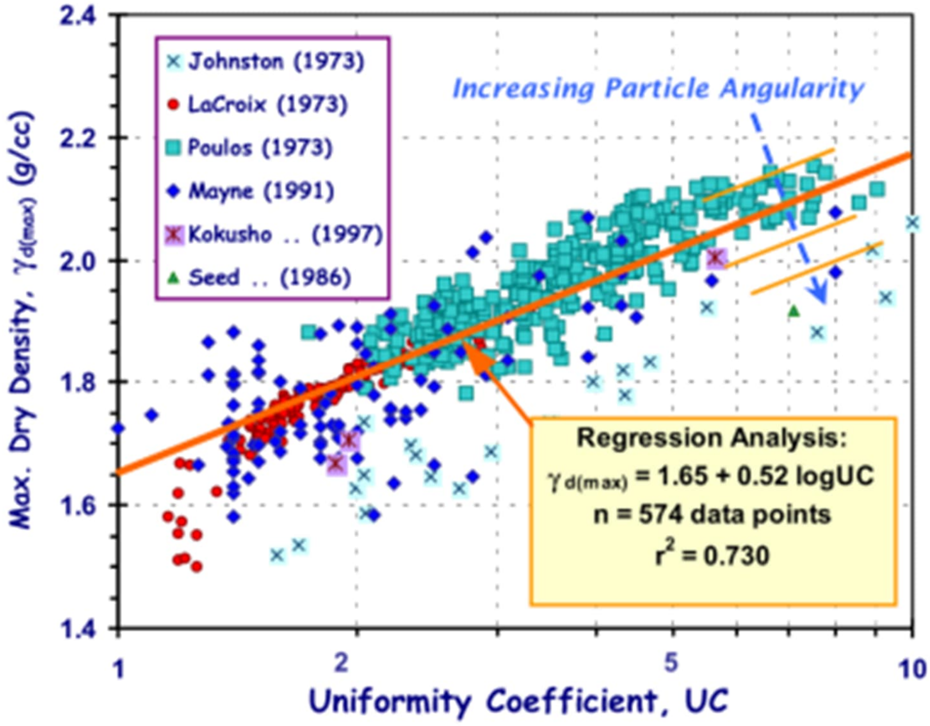


Figure 10: Maximum dry density relationship with sand UC (Mayne et al., 2002)

3.1. HISTORICAL TESTING AND COMPACTION HISTORY

This timeline history for the CBR and Dry Density Ratio (DDR, but also DR) tests are summarised in Figure 11. The modified compaction test evolved in the 1940s with the US Corps of Engineers for "heavy" equipment. This has been wrongly interpreted as a modified compaction test is a better test. The timeline history for the compaction equipment and vehicle loads is summarised in Figure 12. This shows that since the 1940s both equipment and vehicle loads have continued to increase and with no modified – Revision 2 , 3 or 4 compaction test. Industry has accepted a modified test despite changing loads. This logic does not improve the quality of testing.

The weight of vehicles and compaction rollers has increased over the years. Early machines prior to 1900 were 6 tons but increased to 10 to 12 tons by the time the standard Proctor compaction test was introduced in the 1930s, and increased further to 15–20 tons in the 1940s by the time the modified compaction test was introduced. During the early standardisation of tests (1950s) the development of smooth drum vibratory compaction rollers further improved compaction efficiency, with machines around 20-25 tons. This machine weight continued to increase by the 1970s. In the 1990s, advances in engineering and construction materials allowed for larger, more heavy-duty compaction rollers with typical weights of 30-35 tons. The introduction of high-frequency vibratory compaction rollers post 2000, and sophisticated control systems led to larger and heavier models weighing over 40 tons.

Vehicle weights also changed over the years. However, it is truck axle loads that governs road pavement performance. The average truck axle load was around 2.5t in the 1920s increasing to an average of 4.5t in the 1940s and 6.5t by the 1960s.

The average truck axle load further increased to approximately 8.5t by the 1980s with technological advancements in truck design. Truck axle loads vary depending on the type and purpose of the truck. However, standard axle loads of 80kN are typically used in design and the different vehicles used as an equivalent standard axle load (ESAL). Pavement fatigue and deformation are then based on standard axle repetitions (SAR).

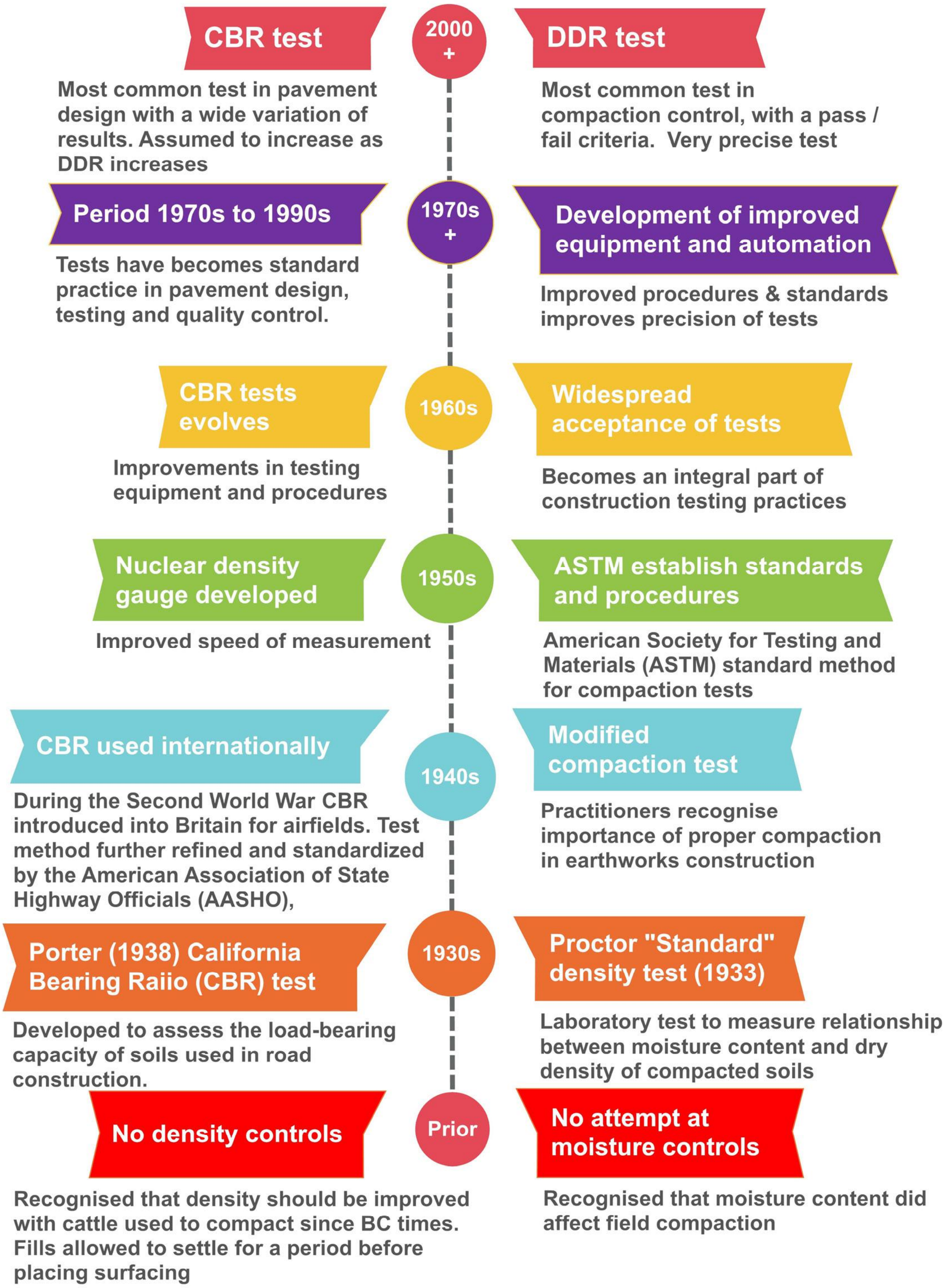


Figure 11: Timeline history of DDR and CBR tests

3.2. EARLY DAYS TO ASSESS BENEFITS OF TESTS

The historical decision as compared to present considerations to use DDR as the key compaction control parameter is discussed in Look (2024a) using force field analysis as a decision-making tool. The high precision of density was recognized in the early days and a governing factor in its implementation in quality control. For example, in the Highway Research Board (1967) symposium on compaction of earthworks and granular bases, Selig and Truesdale (1967) examined the independent and joint effects for:

- M – Moisture level; S – Soil type
- E – Compaction equipment; T – Lift thickness
- C – Compactive effort – Least individual effect

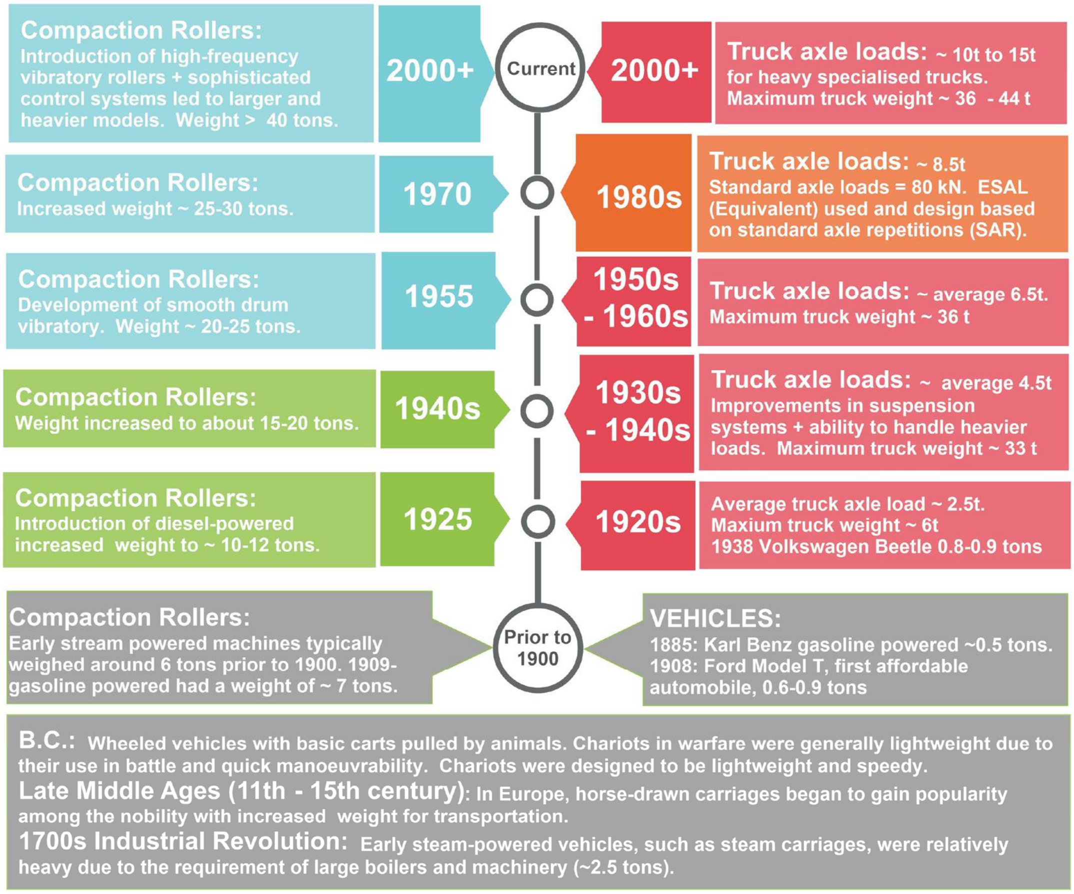


Figure 12: Timeline history of compaction equipment and vehicle loads

Table 3 summarises this variation for the properties measured by Selig and Truesdale (1967). This ranking shows density has a low variability compared to other tests. Moisture content was the most significant factor influencing the strength and stiffness of the soil, but with poor repeatability. Thus, precision took preference over the more useful but wider variability of other soil measurements of the time. DDR was thus elevated to its current prominence due its high precision and not to its superior indicator of key issues. An expanded discussion is provided in Look (2023a, 2023b).

Table 3: Range / average ratio of properties for all effects (Selig and Truesdale, 1967)

Measurement	Range / Average (%)
Dry Density	19
Wet Density	23
Seismic Velocity	75
Moisture Density[1]	87
Plate Load	105
Moisture Content	112
Penetration Resistance	145
Field CBR	177

[1] Moisture Density = Wet density – Dry Density. This term is not commonly used

The relative significance of the various effects was based on growth curves for the test sections with roller coverages showed the bearing plate was influenced the most by E (compactive effort) and M (moisture level).

3.3. RECENT ASSESSMENT OF BENEFITS OF TESTS

Table 3 provided the variability of various tests in the 1960s. In recent times various test instruments were compared to density for five sites (**Table 4**) with the coefficient of variation (COV) shown in Look (2021). The COV represents the range of precision, and the density test is the standout leader (COV = 2.0%) as compared to the Plate load test with a COV of 77%. Most other test (including CBR) have a COV of 25 to 55%. However, precision is the 2nd preference as compared to accuracy (**Figure 3**).

Accuracy was assessed in terms of how well the results compared with each other for similar high, low, and median values for the 5 test sites. The PLT had the highest accuracy as described in Look (2019). The DDR has the lowest correlation with the other tests with dendrogram analysis showing these interrelationships (Look, 2021). The Plate load modulus and DDR had a correlation R^2 value less than 0.15. The same modulus values were measured once the DDR exceeded 90% i.e. 90%, 95%, or 100% DDR had the same modulus for the 3 materials tested.

Given that DDR is the most precise test (**Table 3** and **Table 4**) then how does that relate to strength and modulus? **Table 5** shows different materials have different strength and modulus values irrespective of the same 95% density ratio achieved. A DDR value does not have a direct relationship with the strength of the compacted material and strength is inferred. **Table 5** also highlights that different compaction equipment may produce different friction angles at the same DDR.

Table 4: COV for various tests considered over 5 sites

Test	Coefficient of Variation – COV (%)		
	Median	Low	High
Dry Density Ratio (DDR)	2.0	1.8	2.9
Geogauge	26.5	19.1	34.5
Prima LFWD	33.5	15.0	35.7
LAB CBR	40	17.0	58
Zorn (LFWD)	34.1	21.6	51
Clegg	36.0	26.0	54
PANDA			
(50 – 100mm)	53	34.0	74
(150 – 200mm)	50	48	92
DCP			
(50 – 100mm)	38.0	28.0	97
(150 – 200mm)	53	34.0	74
Plate Load Test (PLT)	77	14.0	142

The COV shown uses only 1 decimal place below 50% and no decimal place above 50%

Table 5: In-situ modulus and strength at 95% DDR (Look, 2021).

Fill Material Origin	PLT E_{V2} (MPa)	In situ angle of friction φ (°)	
		Smooth	Padfoot
Sandstone	70	45	45
Interbedded Siltstone/Sandstone	40	41	39
Basalt	65	39	43

Data for another site is shown in **Figure 13** comparing the Dynamic Deformation Modulus (E_{vd}) with DDR. No relationship is evident. The E_{vd} is the stiffness modulus of the soil under a dynamic load, calculated from the deflection measured during a light falling weight deflectometer (LFWD) impact. It is conceptually similar to a plate load test modulus, but is dynamic and not static and is a quick and easy test, making it ideal for compaction control.

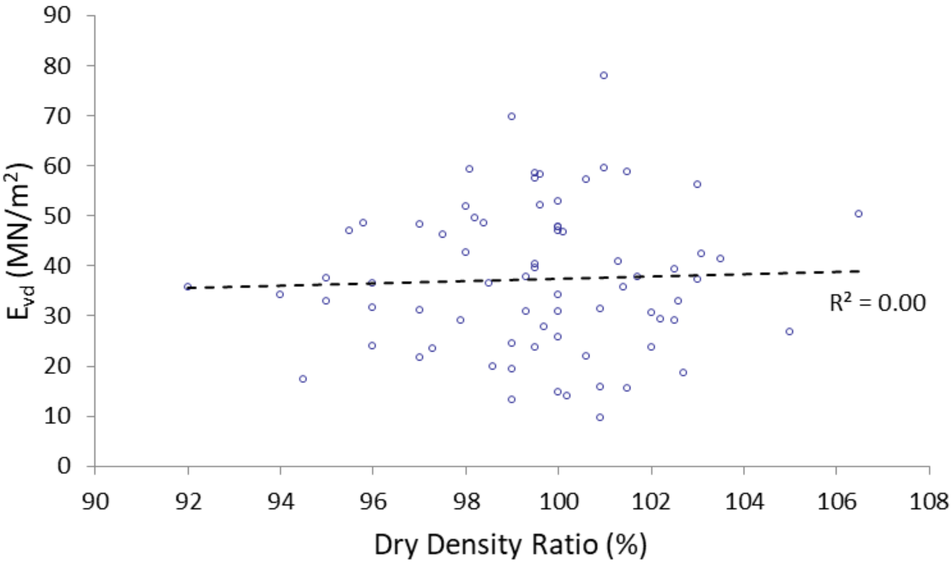


Figure 13: Dynamic Deformation Modulus (E_{vd}) versus Dry Density Ratio

The high precision of the DDR test is contrasted with its low accuracy in predicting strength or modulus values. This shows design input and construction quality have a disconnect. We incorrectly assume a high DDR produces higher strength or modulus. The aim of compaction is to reduce the air voids and typically air voids less than 10% would occur at 90% DDR. The line of optimums (LOC) for varying compaction energy is the true target, as lab compaction energy is well defined but compaction energy is unknown in the field. The LOC would be between 10% to 5% air voids.

In temporary working platforms where high construction equipment loads are used, an engineer is ill-advised to rely on density only as indictive of a strength or modulus.

Density ratio on its own should be paired with additional testing, as it is not standalone and not a performance indicator. The CBR is commonly the paired test but has many associated implications of errors in testing (**Figure 14**). Look (2019) showed density ratio reporting from 5,619 quality control results occurred 13 days (median) to 22 days (mean) after sampling. No contractor is waiting 2 – 3 weeks on results to continue placement of the next lift of a fill, as such standby times of equipment has associated project costs and delays. It is most likely testers provide verbal reporting, with quality testing certification lagging.

Density ratio testing is then becomes more a tick box approach to show quality tests have been carried out, rather than as a rigorous quality decision tool. Similarly, for 746 CBR tests were undertaken during construction. Test results were 34 days (median) to 43 days (mean) after sampling has occurred (Look, 2019). This time lag is not a quality assurance process and often results in contractual argument after the fact of placing additional fill. Surely we can do better than using these lag indicators.

Using the MDD and OMC as a reference point the CBR is carried out. The associated error (from test repeatability and reproducibility) is highlighted in **Figure 15**. The correlation error is also shown. During construction reliance on DR alone has associated uncertainties (**Figure 13** and **Table 5**) as a high densities does not necessarily mean high modulus.

3.4. EFFECT OF OVERSIZE

Aside from the poor timing of the tests, one still has the issue associated with the quality and interpretation of the results. The density testing using mould A is not valid if more than 20% of material is retained on the 19mm sieve. A check of 189 test results show 18.5% invalid reported results (**Figure 16**) with less than 80% passing the 19mm sieve according to AS 1289.5.1.1 (2017). In addition, a significant number of tests (31.5% of test results) should have a correction applied to the MDD and OMC, or should use mould B which allows up to 37.5mm size. Yet those compaction test results show no corrections applied. The gradings were done independently of the compaction tests, with test certificates showing "no oversize". (Note these results are over 10 years ago – and possibly improvements may have occurred since that time).

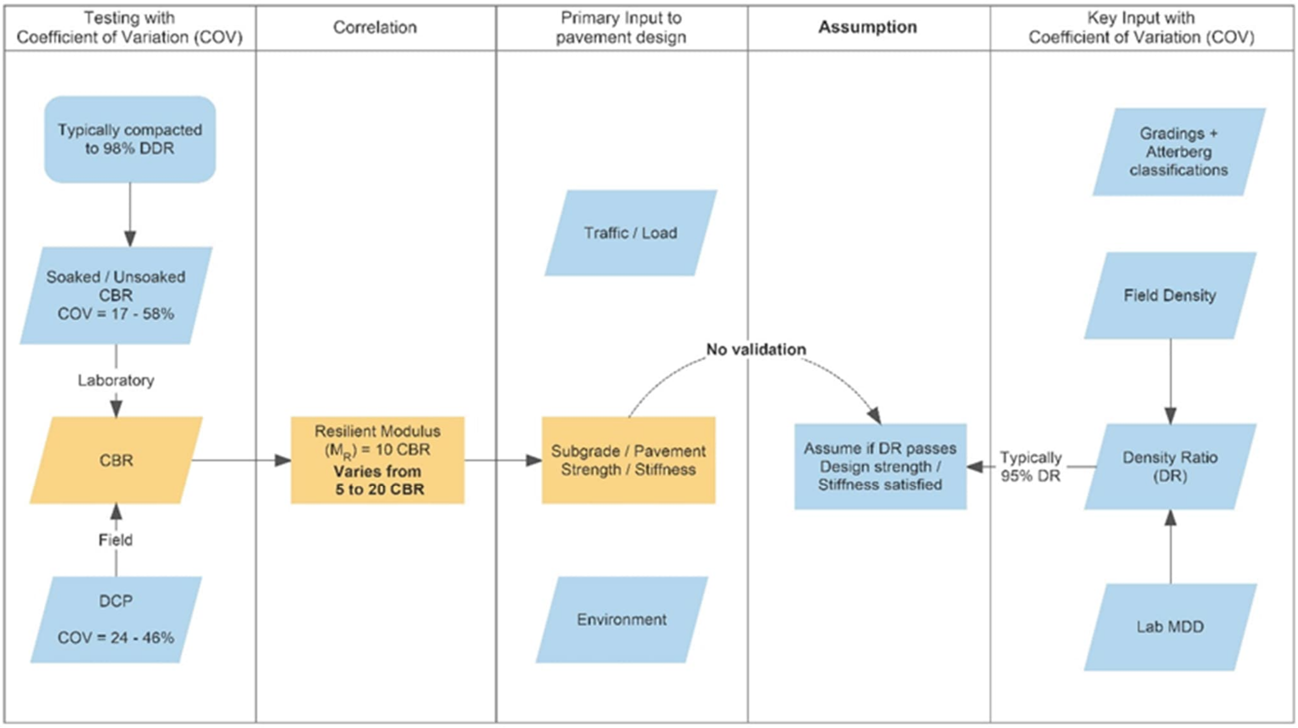


Figure 14: Key input vs implicit errors in current approach of testing 2nd or 3rd order parameters

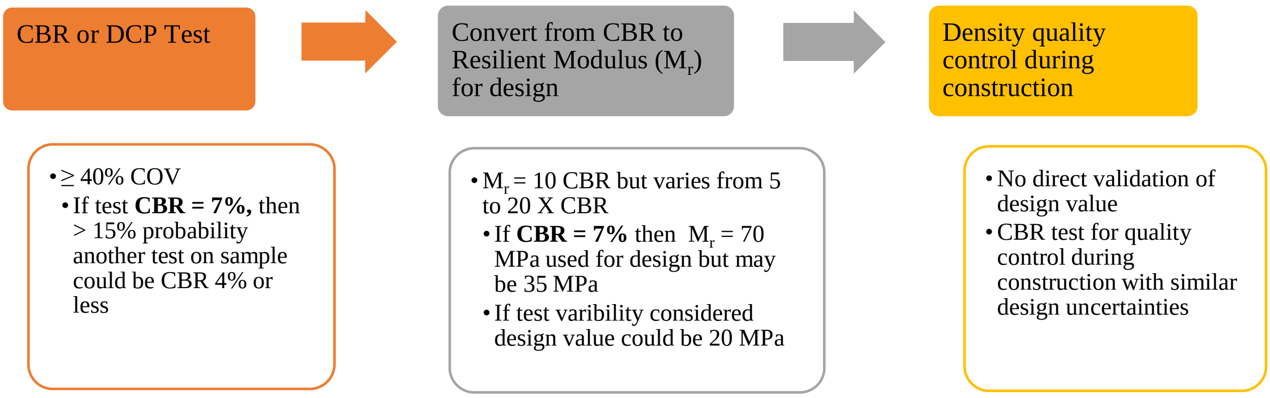


Figure 15: Industry accepted testing errors

Accounting for oversize is not new to standards. BS1377 (1975) required accounting for the percentage passing the 20 mm sieve. (Note BS uses "true" metric numbers such as 20mm while AS uses numbers such as 19mm and 37.5mm – which are "conversions" using U.S. sieve sizes).

Gravel contents greater than 20% to 30% affect compaction. When the gravel content is greater than 60% to 70% the voids are not filled and leads to reduction of the maximum dry density of the total material (Farrar, 2006). When soil contains greater than 30% larger than 19mm, a method specification should apply.

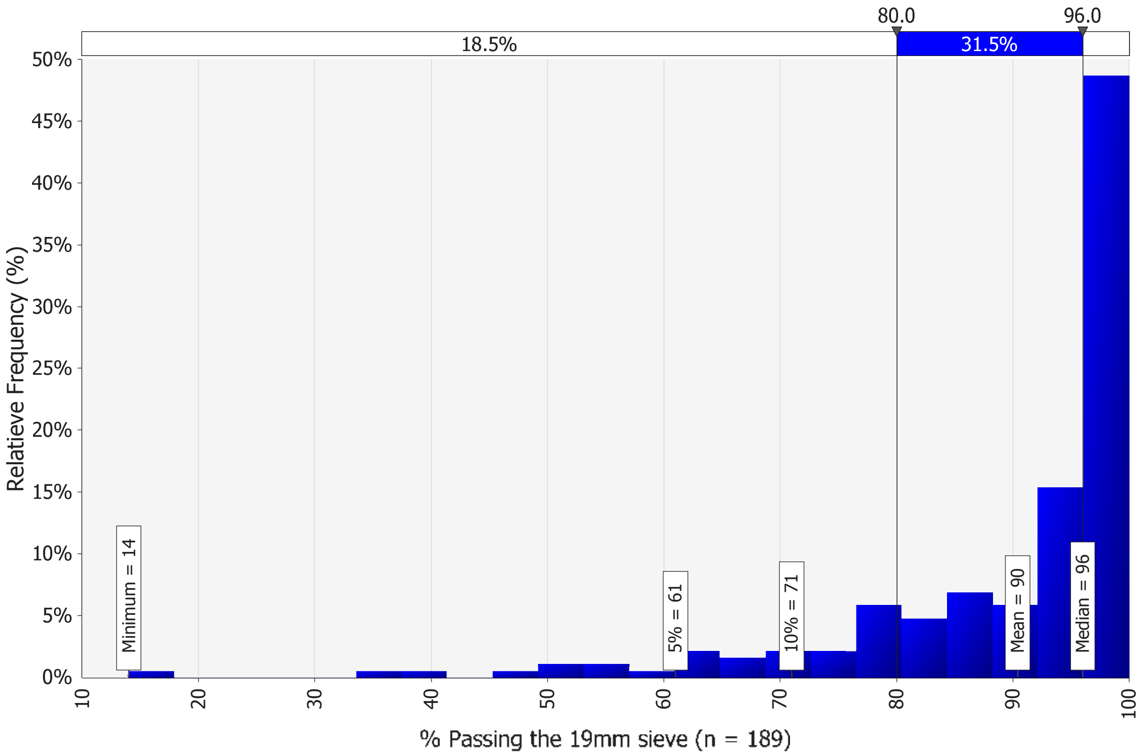


Figure 16: Density test using mould A requires over 80% of soil to be passing 19mm sieve. For this "passed" project 18.5% did not meet the test standard

4. RELEVANCE OF CBR DATA

The CBR tests were identified not to be trusted (**Figure 1**). The **CBR is an index test** that has evolved into a key design parameter for subgrade assessment. The CBR is not a fundamental soil property but enables the formulation of empirical correlations. Changing procedural aspects may lead to change in the test result. For example, curing periods or surcharge weights may alter the test results. A few aspects of the CBR testing are:

- CBRs < 8% have poor reproducibility i.e., comparing clay with rock; CBR > 100% test result can be obtained, but do not use.
- The test has a large variability; Rallings (2014) showed less than 60% of results are within ± 30% of the median value when multiple laboratories are compared with the same samples in assessing reproducibility of tests on similar samples at different laboratories.
- The test can use a soaked or unsoaked condition. A soaked CBR test is over-conservative in arid environments.
- A swell value is measured at the end of soaking and is also an important assessment parameter. For low CBR values, the swell value is arguably more important than the CBR test value.
- A 4.5 kg surcharge is used during the soaked test; this mass should be varied to be representative of the overlying material; both the CBR and swell values may be affected (AS 1289.6.1.1 (2014)
- The Equilibrium Moisture Content (EMC) is more important than testing at the OMC (Look, 2005).

Some of the underlying assumptions associated with this test will be discussed to show the effect of not following procedures. Discussion points includes:

- One point versus multi point CBR
- Soaked versus unsoaked and soaking time
- Effect of surcharge
- Curing preparation

- CBR dependence on the many other input parameters such as compaction density ratio (DR), swell or moisture ratio after soaking. This will be examined using:
  - Dendrogram analysis
  - Pearson correlations
  - Sperman rank correlation
- Field versus laboratory CBR
- Field values compared with other types of tests

Most engineers believe that the peak CBR occurs at the OMC (**Table 1**). This has the led to many one point CBR tests where the CBR at MDD / OMC is used. This interpretation error will be first discussed as it is so commonly used.

4.1. ONE POINT VERSUS MULTI POINT CBR

Engineers assume a direct relation between density and strength or modulus (i.e., the greater the density, the higher the modulus of the compacted material - **Figure 9**). However, the peak CBR does not necessarily occur at the MDD. Seed and Chan (1959) had shown the peak strength is lower at the MDD and is higher at the lower moisture content even though the density is lower (**Figure 17**). The key take away from this figure is that we have **known this since 1959**.

The peak CBR not being coincident with the OMC / MDD was shown in Look (2023b) for a granular material with its peak strength dry of optimum (below MDD) and for a CH clay with its peak CBR wet of OMC. Thus, using a one point CBR value, while commonly used in industry is not an indicator of peak CBR value. Using a CBR at OMC is likely to be conservative, and therefore errs on the “correct” side. The above discussion is for a soaked test. An unsoaked test would have its maximum CBR dry of OMC.

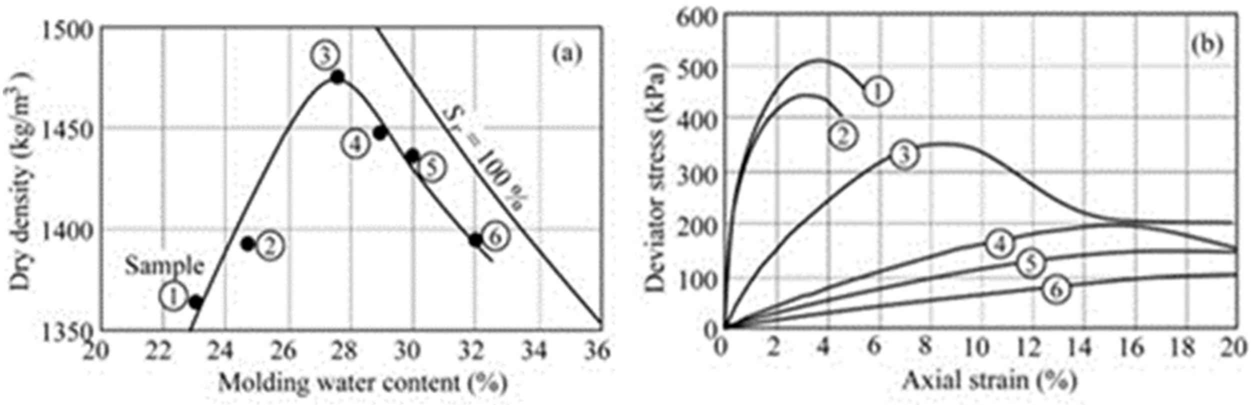


Figure 17: Influence of moulding water content on the dry density (a) and stress-strain relationships (b) for compacted samples (Seed and Chan, 1959; here from Leroueil and Hight, 2013).

4.2. SOAKED VERSUS UNSOAKED RESULT

There are many instances when a soaked result should not apply, although that is often a de-facto approach in industry. An unsoaked conditions should apply to material with:

- < 15% fines
- Excellent drainage
- Low rainfall environments (< 500 mm annual rainfall)

The soaked condition is representative of a 4-day flood which can produce a reduced strength but is not necessarily the design value. In Australia, a 4-day flood is not representative of extreme flood events and 7 or 10-day soaked tests are more appropriate for soaked conditions for low permeability materials.

The significant effect of soaking is shown for a site at Ipswich, Queensland, with a Weighted Plasticity Index (WPI) of above 5000 – an extremely reactive clay site. The unsoaked CBR is significantly higher than the soaked test with little difference in CBR between the 4 day and 7 day soaked tests (**Figure 18**). However, for a high WPI material the movement and not the strength governs the design. The CBR swell is significantly different for the 10 day versus the 7 day soaked test value (**Figure 19**).

This site has a history of flooding and the unsoaked CBR is inappropriate. The moisture content (MC) at the top 25mm depth of soaked sample is more highly correlated to CBR (**Figure 20**) than the MC of the remainder of the sample (**Figure 21**) with correlations R^2 of 0.43 and 0.17, respectively, for the 4 day soaked value. The correlation improves for the 10 day soaked value and suggests the sample had not yet reached its full "soaked" value at 4 days.

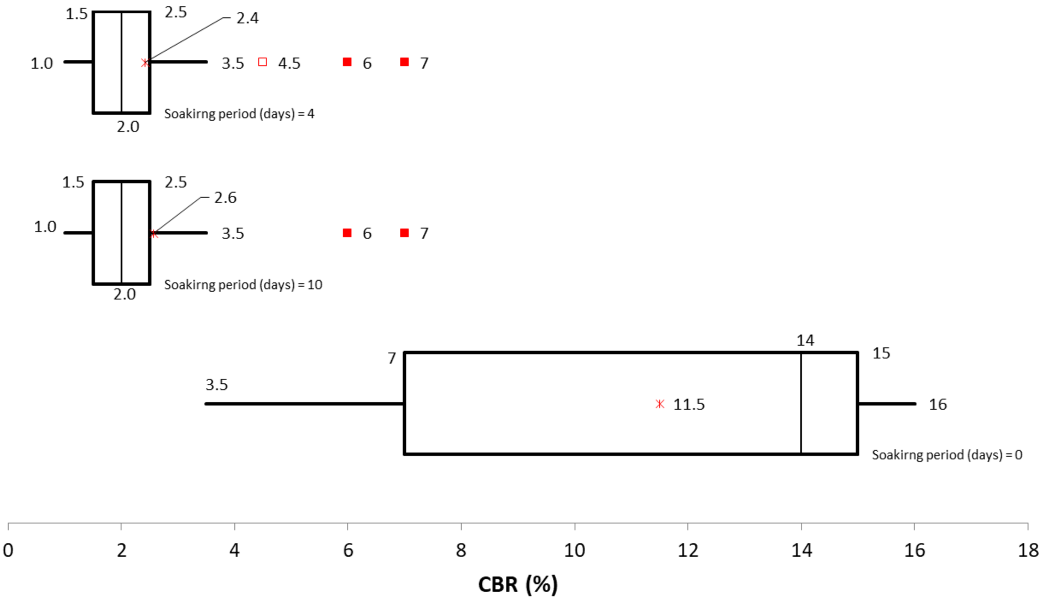


Figure 18: Effect of soaking on CBR for a high WPI material

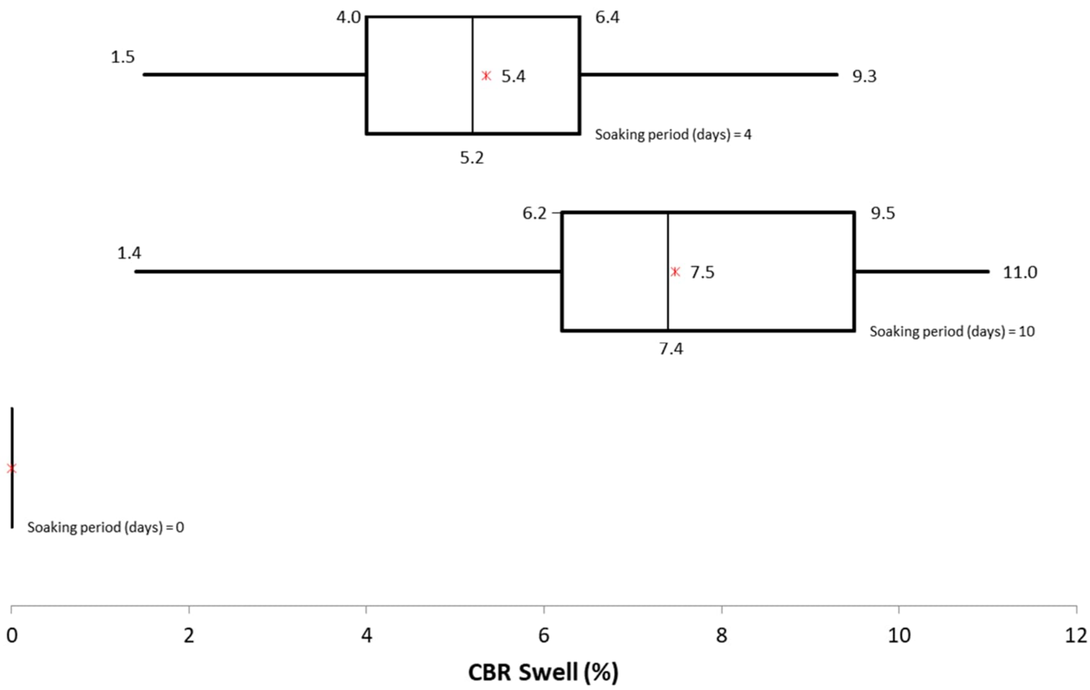


Figure 19: Effect of soaking on the CBR swell for a high WPI material

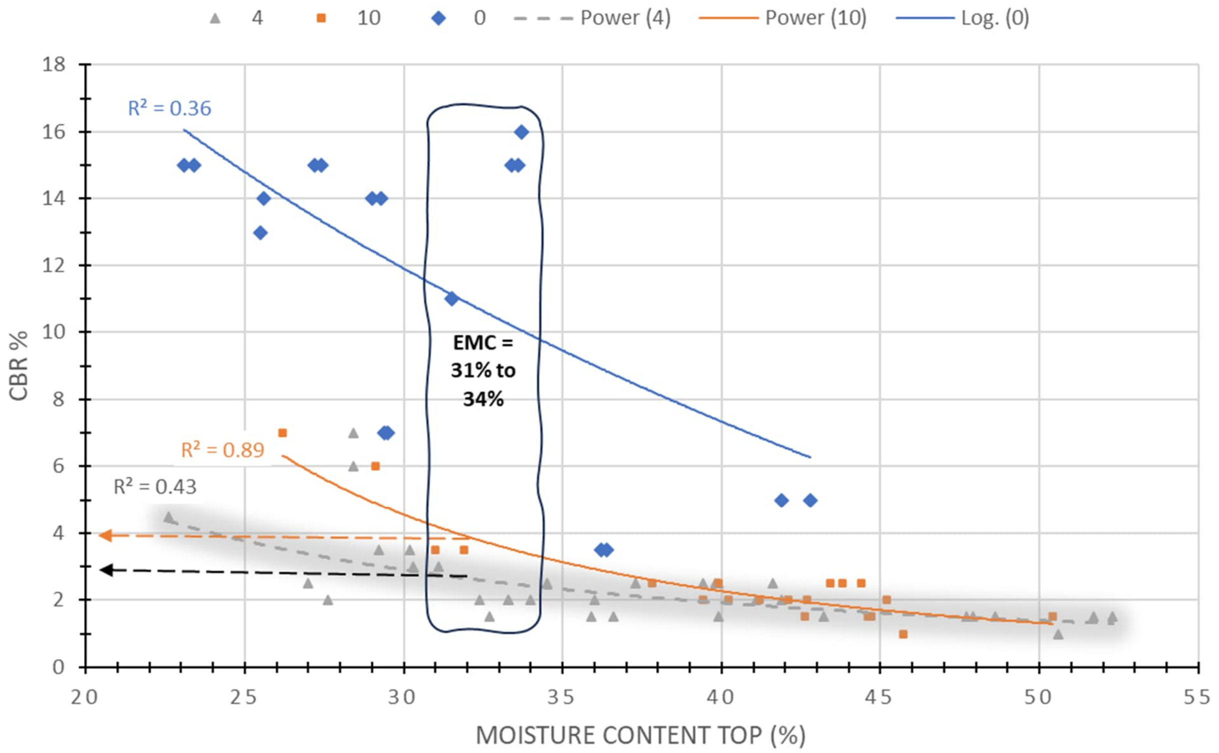


Figure 20: Selecting the design CBR value based on EMC of 31% to 34%. Top 25mm MC of sample applies

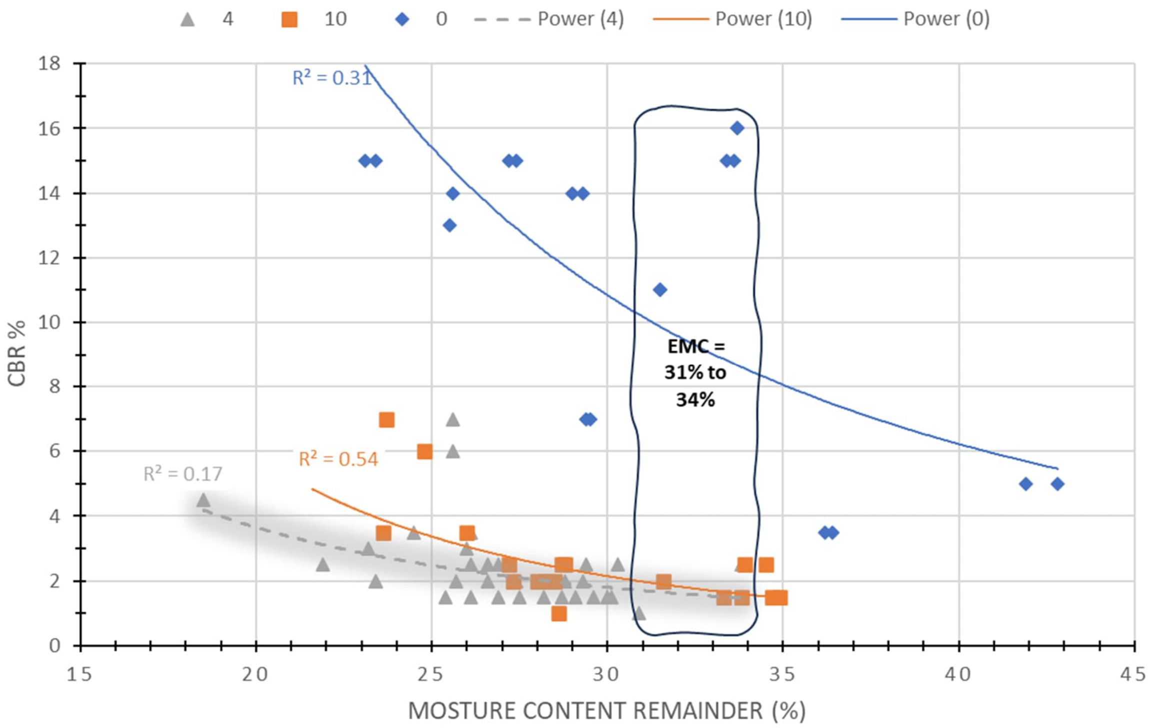


Figure 21: Selecting the design CBR value based on an EMC of 31% to 34% at this site. MC of sample shown

Selecting an appropriate CBR test is no trivial matter, yet the majority of reports sighted simply used a 4 day soaked CBR (value of 1.5% at the lower quartile value) at the OMC (23%). The steps illustrated in **Table 6** show the rationale and associated errors for an over simplified approach using a 4 day soaked CBR test only. Comparisons show issues with selecting:

- Soaked / unsoaked or 4 day / 10 day soaked
- EMC vs OMC
- MC of rest of sample vs top of sample

Table 7 shows the equivalent steps and rational for selecting a design swell value.

Table 6: Steps associated with obtaining a design CBR value for this high WPI site

Soaking Test	CBR (%) at EMC = 31% to 34%		CBR at OMC = 23%		Comment
	MC of top 25mm of sample (Figure 20)	MC of remainder of sample (Figure 21)	MC of top 25mm of sample (Figure 20)	MC of remainder of sample (Figure 21)	
Unsoaked	11%	9.0%	16%	17%	Inappropriate as site has a history of flooding
4 day	3.0%	1.5%	4.0%	3.0%	Inappropriate as inundation exceeded 4 days
10 day	**4.0%**	1.5%	8.0%	4.0%	Appropriate CBR – but MC of top of sample governs. The CBR at OMC is not appropriate

Table 7: Steps associated with obtaining a design swell value for this high WPI site

Soaking Test	Swell (%) at EMC = 31% to 34%		Swell at OMC = 23%		Comment (Figures not shown)
	MC of top 25mm of sample	MC of remainder of sample	MC of top 25mm of sample	MC of remainder of sample	
Unsoaked	0%	0%	0%	0%	Inappropriate as site has a history of flooding
4 day	3.5%	6.5%	0.5%	3.0%	Inappropriate as inundation exceeded 4 days
10 day	**5.0%**	9.0%	0.0%	4.0%	Appropriate CBR – but MC of top of sample governs. The CBR at OMC is not appropriate

Interestingly the extended 10 day soaked test provided a higher CBR value than the 4 day soaked test. However, as indicated previously, the CBR swell (movement) governs the design and not the subgrade strength. The improved correlated swell at 10-days soaking were also evident with higher swell values at 10-days (**Table 7**).

The tables compare the many factors to be considered in selecting a design subgrade value from a CBR test. For the 12 factors considered, there were 3 combination of factors for which the CBR = 4%. One may "luckily" still have the appropriate CBR at the OMC in a 4 days soaked – despite that model not being appropriate. However, as the swell governs at such high WPI then the designer would have underestimated the ground movement.

4.3. EFECT OF SURCHARGE

The Australian Standards for determination of CBR for soil allows for a change in surcharge weight depending on the layer thickness above. Anecdotal discussions with laboratories suggest that over 99% of testing is caried out on surcharge of 4.5 kg only. This surcharge would be appropriate for a layer thickness less than 200mm above the subgrade.

A low compressibility silt (A-4 soil) and a granular material with fines (A-2-4) was examined by Khalid et al. (2022). The results show an increase in CBR value with a higher surcharge (**Figure 22**).

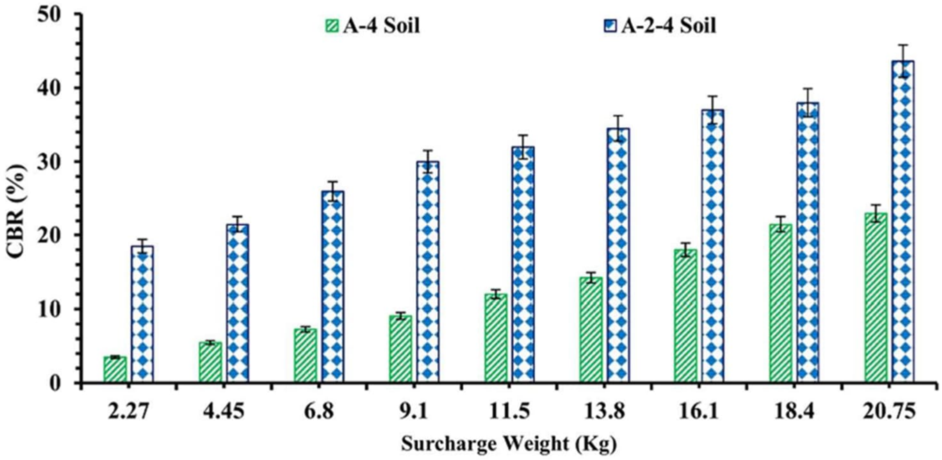


Figure 22: CBR compared to different surcharge weight (Khalid et al., 2022)

Using the data of **Table 7**, the effect of surcharge on the swell values is show in **Figure 23**. At the EMC, the swell change is minor, from 4.0% to 3.5% for the surcharge changing from 4.5 kg to 18kg, respectively. This is for a 4 day soak, and based on the trends of **Table 7**, the swell may be larger for a 10 day soak.

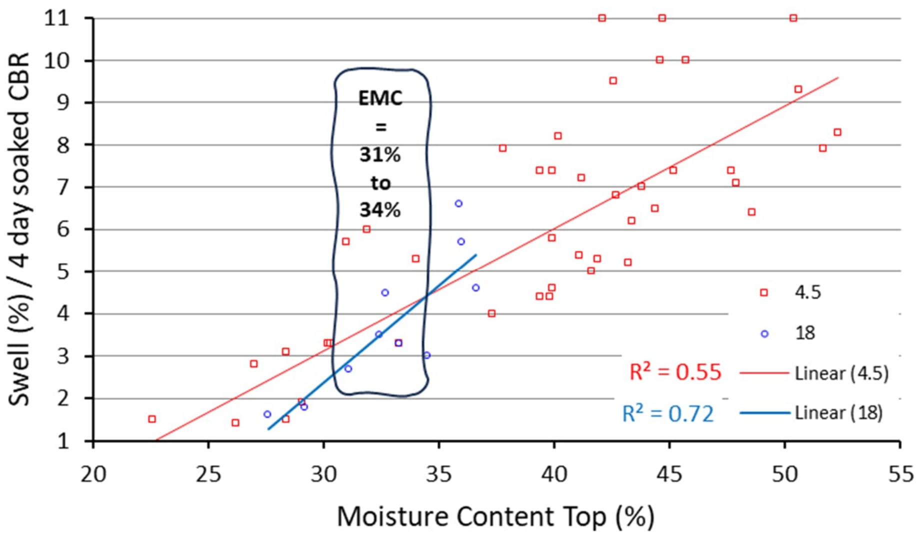


Figure 23: Effect of surcharge weight (4.5kg and 18 kg) on CBR swell at 4 day soaked

4.4. CURING PREPARATION

An example of the effect of curing time is shown for a Bundamba clay with a plasticity index of 47% and grading with 97% passing the 425-micron sieve and 69% to 82% fines (Look, 2021). Samples from the site were prepared and cured for 0, 1, 4 and 7 days, compacted, and the CBR (soaked and unsoaked tests) determined at the various times. The following observations were made from compaction and CBR tests on these samples:

- Curing for 0 to 1 days produced higher MDD values than 4 to 7 days. This leads to more compaction effort in the field being required, or the *in-situ* material is more likely to fail the density ratio test (**Figure 24**).

- Lower curing days produced higher CBR swell values which incorrectly required more remove and replace of material than is necessary for limiting reactive soil movements of pavement subgrades. This leads to increased construction costs. There is a significant difference in swell values depending on whether the initial sample was wet or dry i.e., the field moisture content at the time of sampling influences the soaked CBR swell test results when there is inadequate curing.
- A curing time of 0 days produced a lower CBR (soaked) value. This results in over-design of the pavement (**Figure 25**).

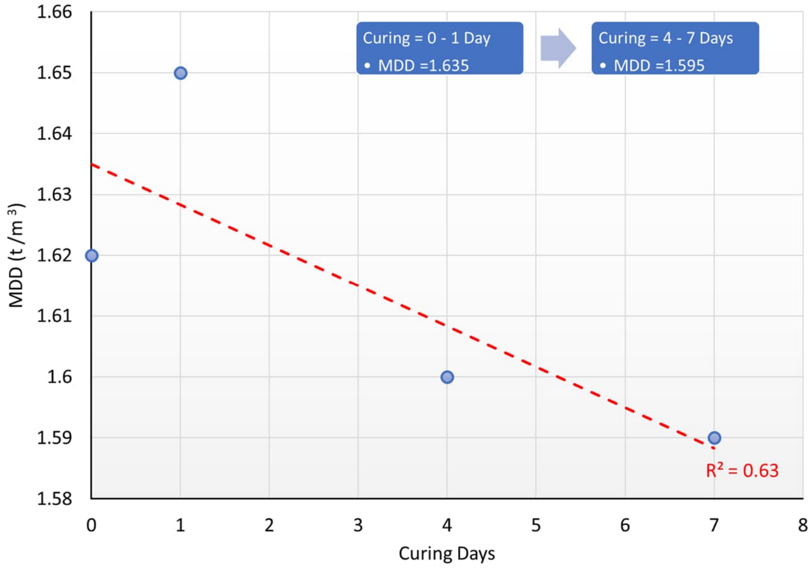
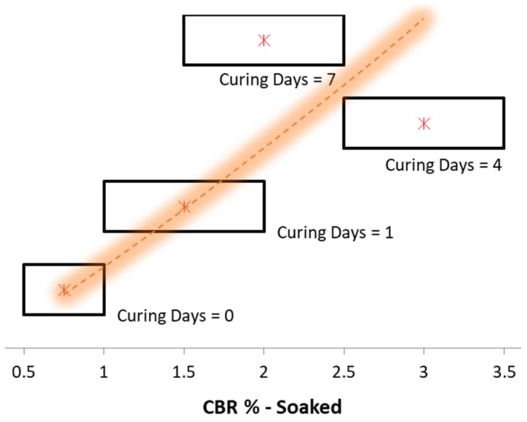
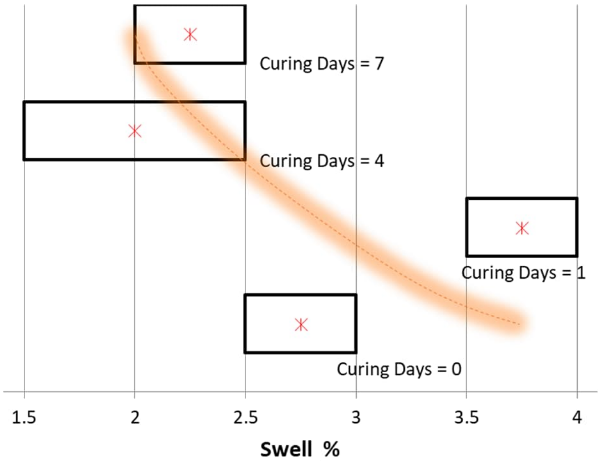


Figure 24: Curing – effect on MDD



(a) Soaked CBR (Lower cure → Lower CBR)



(b) CBR Swell (Lower cure → Higher swell)

Figure 25: Curing – effect of soaked CBR

High plasticity clay samples require 4 to 7 days curing time according to Australian Standards. Over the years I have observed many NATA laboratory test reports not satisfying this procedural test requirement. Curing requirements for the granular or low plasticity material requiring 1 to 2 days curing, would typically be satisfied. These unnoticed procedural aspects have both design and construction implications.

4.5. DATA SCIENCE AND PREDICTIVE STATISTICAL MODELS

Making predictions from geotechnical data requires grouping to generate valuable insights. The Pearson linear correlation (as used in EXCEL) is the most common approach and assumes a bivariate relationship. The primary goal of analyzing bivariate data is to understand the relationship between the two variables. Scatterplots are commonly used to visualize bivariate data, and the correlation coefficient is often used to quantify the strength and direction of the linear relationship between the variables The Pearson correlation assumes a normal distributed relationship and is sensitive to outliers. The Spearman Rank order correlation is used when the bi-variate data distribution is non-normal or when the data is ordinal level. The data is replaced by their ranking, and a parametric correlation is found between the ranks for a rank correlation coefficient. Geotechnical data is often non-normal and has outliers and the Pearson coefficient used in EXCEL is often not appropriate. Both the Pearson and Sperman coefficient analyses are used on construction quality data to illustrate the pitfalls in only using the EXCEL approach

Multivariate analysis with complex and multiple data relationships requires looking beyond paired correlations. The multiple relationship factors can be shown by Principal Component Analysis (PCA) or dendrogram analysis (Look, 2021, 2025c). Hierarchical clustering is used on test data to predict groupings within the data set by generating a link between each single observation and its nearest neighbour. Those distances between parameter values are then used to predict sub groups within a data set. A dendrogram is a visualisation tool that show the similarities and branching between groups in a tree like structure. By clustering similar data points together, dendrogram analysis can help geotechnical engineers gain insights into the underlying relationships present in the data.

4.6. DENDROGRAM AND CORRELATION ANALYIS OF CBR TESTS

Dendrogram and PCA analysis application in geotechnical engineering are shown using CBR test data in Look (2021, 2025c). The CBR is used in design and (incorrectly) assumed to be most closely related to the compaction density which is used in earthworks quality control testing. The dendrogram analysis of a CH clay material (55 data sets) for the relationship factors is shown and visualised with dendrogram analysis (**Figure 26**) using nonparametric statistical tests. This analysis shows the after-swell moisture ratio governs the CBR test value, and the density ratio at compaction is least related to the CBR value. The moisture related parameters (degree of saturation (DOS), moisture ratio (MR) at compaction) are most related to the CBR while, the density related parameters (Density ratio (DR) at compaction, Dry density(DD) and maximum dry density (MDD)) are least related.

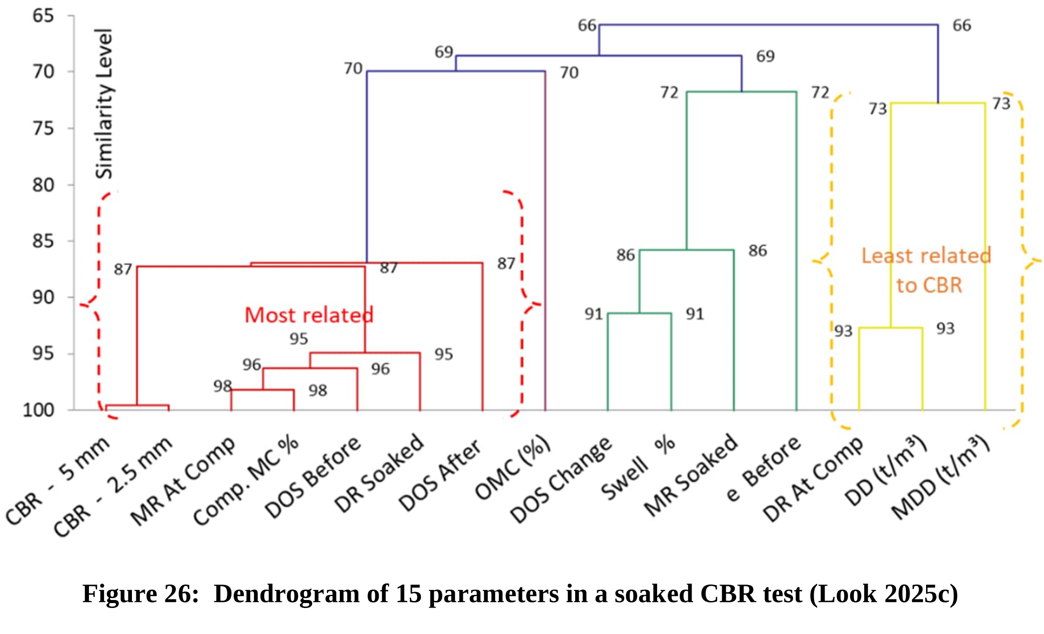


Figure 26: Dendrogram of 15 parameters in a soaked CBR test (Look 2025c)

This visualization provides simplified insights that may have been achieved by carrying out multiple plots such as **Figure 27** for the relationship of CBR with the density and moisture ratio (MC/OMC) when compacted and after soaked, respectively. The data noise and non-normal distribution is evident which fails to show the many associated parameters affecting the CBR value. This data shows the poor relationship of the soaked CBR index (strength indicator) and the compacted density ratio (DR). Yet the DR is the key quality control parameter used to assess strength during construction material testing. By adopting such hierarchical clustering analysis, data noise is removed and key risk factors for geotechnical modelling can be identified.

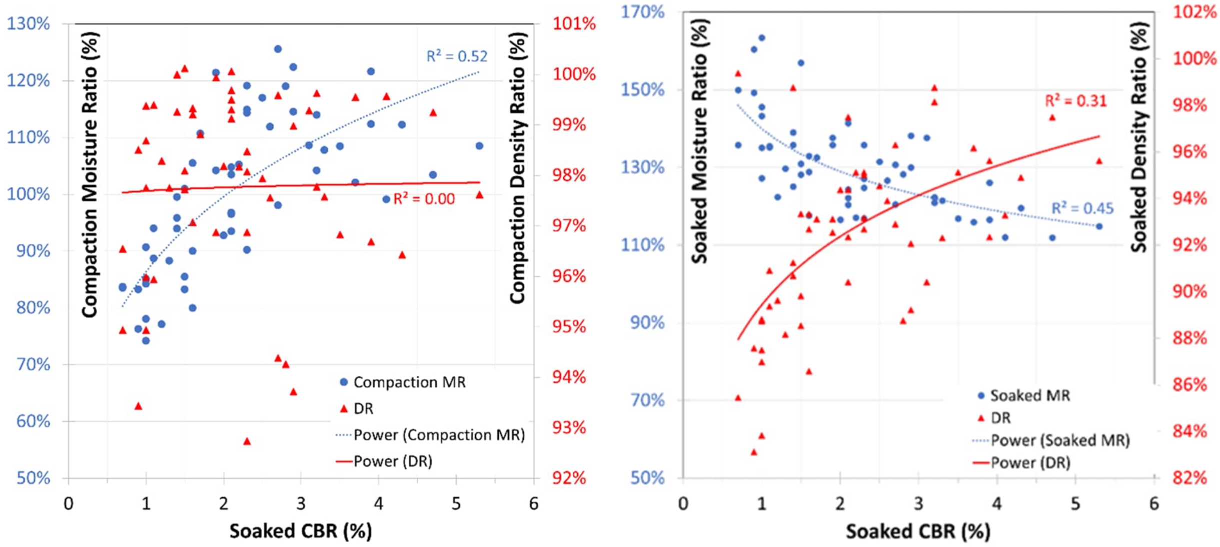


Figure 27: (a) Soaked CBR vs compaction MR and DR (b) Soaked CBR vs after soak MR and DR

Many of the parameters are derived while some are measured results. An expanded data base (172 data sets), which includes the above Cooroy clay as a subset, and rationalised dendrogram analysis (removing Dry Density and MDD as it is part of DR) is shown in **Figure 28**. This expanded data analysis shows that the CBR is most related to the DOS before soaking and the moisture ratio at compaction, but is least related to the DR at compaction.

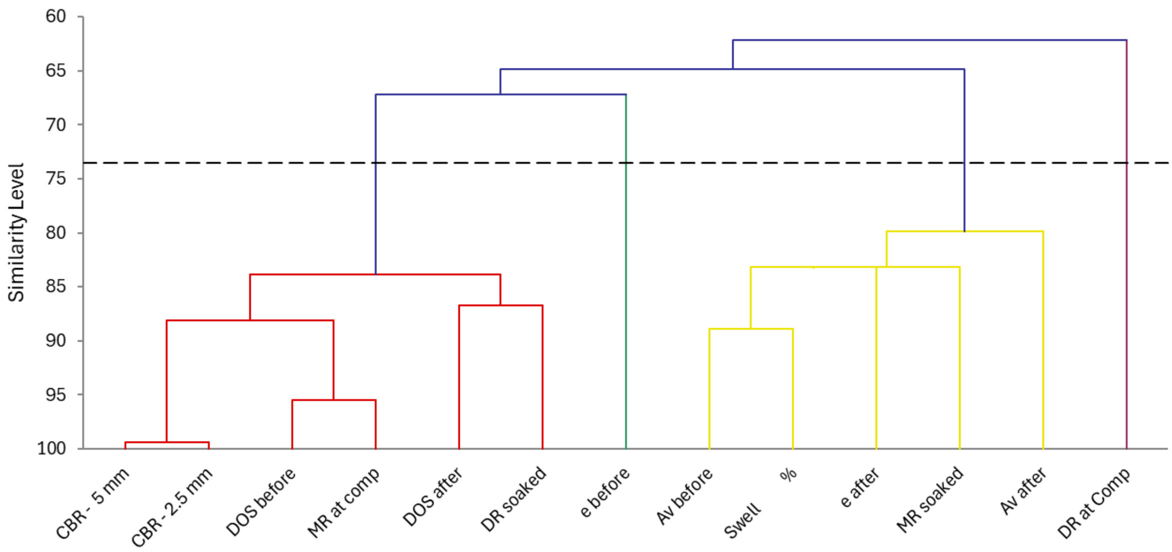


Figure 28: Dendrogram of 13 parameters in a soaked CBR test (n= 172)

This unsupervised[1] dendrogram analysis was compared with the more traditional Pearson and Spearman correlation matrices in **Table 8** and

[1] Supervised algorithms such as classification and regression have the labels to train the model. Unsupervised models such as clustering, PCA or dendrogram analysis has unlabelled data without target variables. The model's goal is to discover hidden patterns, structures, or relationships within the dataset without explicit guidance.

Table 9, respectively. These tables show the parameters of MDD and DD for completeness, although they are not shown in the dendrogram for simplicity. Disappointedly, the three methods do not all provide similar results but it is noted that PCA and Dendrogram are multivariate analysis, while paired correlations accepts data noise. The analysis shows following observations:

- Dendrogram analysis (**Figure 28**)
  - The CBR is most related to the interaction of the DOS before soaking, and the MR at compaction. The MR at compaction is derived from the compaction moisture content and OMC (not included)
  - The CBR is least related to the density ratio at compaction noting that this parameter is derived from the maximum dry density and dry density (not shown)
- Correlation matrix using Pearson (**Table 8**)
  - The CBR is most related to the average moisture content after soaking, the void ratio before soaking and the dry density at compaction
  - The CBR is least related to the density ratio at compaction, which matches the dendrogram analysis. The air voids before compaction also had little effect on the CBR

Correlation matrix using Spearman Rank (

- **Table 9**)
  - The CBR is most related to the swell and moisture ratio when soaked
  - The CBR is least related to the moisture ratio and density ratio at compaction. Yet these are the 2 main parameters in quality control.

The <u>one constant</u> in all 3 analyses is the density ratio at compaction had the least effect on the CBR value after soaking.

Industry practice (implicitly) associates a higher strength with a higher density, as strength or modulus is not traditionally measured in quality control. This is a reasonable assumption as seen in **Figure 29** for CBR vs DD. When the density ratio is used (combining DD with MD) there is no relationship (**Figure 30**). This is evident in both the Pearson and Spearman ranking of **Table 10** and the dendrogram of **Figure 28**.

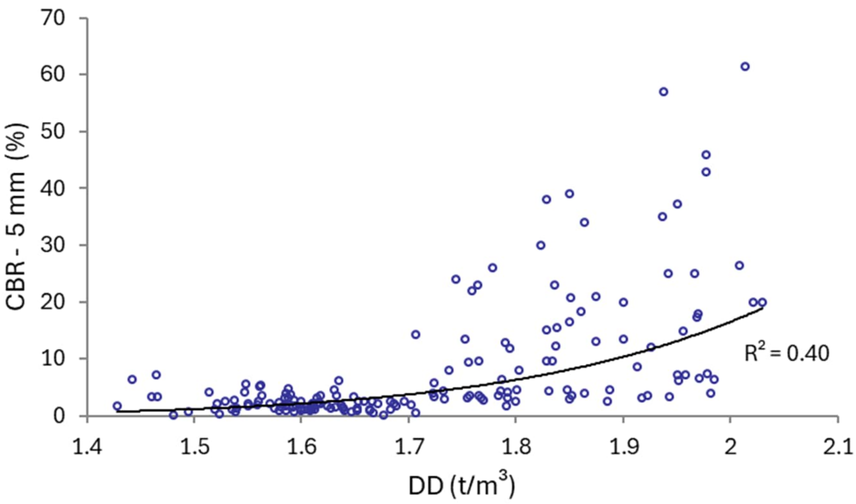


Figure 29: CBR is dependent on the dry density (n= 172)

Because a high strength is associated with a high density, industry has incorrectly applied an inverse logic of a low density implies a low strength. The latter has a partial truth and applies only to material which are inherently strong. The higher CBR material requires compaction to 95% to 101% for DR to achieve its inherent strength properties (**Figure 31**). This should not be extrapolated to mean a higher DR has a higher strength. A low strength material (CBR < 10%) will not achieve a higher strength at high levels of compaction (say 98% DR). Yet a 98% DR of such low strength material is assumed (incorrectly) to be stronger than a 95% DR.

The following density ratios are required to be achieved for different CBR material(**Figure 31**):

- High CBR > 10% : DR = 94.5% to 100.5%. Aside from one data point this could be 96% to 100.5%
- Low CBR < 10% : MR = 92.5% to 100.5%

Table 8: Correlation matrix between parameters using Pearson correlation

Linear Correlation	Comp MC %	DD (t/m³)	OMC (%)	MDD (t/m³)	CBR - 2.5 mm	CBR - 5 mm	Swell (%)	Avg MC after soak	DD (t/m³) after soak	MR at comp	MR soaked	DR at Comp	DR soaked	e before	e after	Av before	Av after	DOS before	DOS after
Comp MC %	1.00																		
DD (t/m³)	-0.84	1.00																	
OMC (%)	0.85	-0.89	1.00																
MDD (t/m³)	-0.86	0.98	-0.91	1.00															
CBR - 2.5 mm	-0.49	0.53	-0.46	0.51	1.00														
CBR - 5 mm	-0.54	0.60	-0.50	0.58	0.97	1.00													
Swell (%)	0.27	-0.56	0.54	-0.57	-0.46	-0.49	1.00												
Avg MC after soak	0.66	-0.95	0.84	-0.95	-0.69	-0.74	0.67	1.00											
DD after soak	0.41	0.27	-0.05	0.21	0.46	0.45	-0.65	-0.74	1.00										
MR at comp	0.37	-0.07	-0.15	-0.09	-0.17	-0.21	-0.34	-0.11	0.52	1.00									
MR soaked	-0.05	-0.26	-0.12	-0.26	-0.39	-0.40	0.59	0.42	-0.59	0.20	1.00								
DR at Comp	0.06	0.26	0.06	-0.07	-0.01	-0.08	0.10	-0.25	0.19	0.03	-0.09	1.00							
DR soaked	0.63	-0.10	0.36	-0.30	0.52	0.51	-0.63	-0.45	0.87	0.55	-0.68	0.17	1.00						
e before	0.81	-1.00	0.91	-0.96	-0.66	-0.71	0.49	0.94	-0.28	0.02	0.23	-0.43	0.09	1.00					
e after	-0.42	-0.29	0.06	-0.21	-0.47	-0.46	0.66	0.75	-1.00	-0.53	0.60	-0.20	-0.87	0.29	1.00				
Av before	-0.56	-0.04	-0.07	-0.00	0.03	-0.01	0.62	0.20	-0.64	-0.75	0.41	-0.14	-0.67	0.04	0.65	1.00			
Av after	-0.47	0.33	-0.27	0.34	-0.17	-0.18	0.36	-0.08	-0.61	-0.39	0.17	0.13	-0.77	-0.33	0.60	0.35	1.00		
DOS before	0.71	-0.18	0.26	-0.21	-0.24	-0.19	-0.47	0.02	0.62	0.75	-0.32	0.07	0.68	0.17	-0.64	-0.97	-0.37	1.00	
DOS after	0.44	-0.36	0.28	-0.35	0.13	0.14	-0.22	0.12	0.58	0.35	-0.13	-0.16	0.74	0.35	-0.56	-0.30	-1.00	0.32	1.00

Table 9: Spearman rank order correlation matrix between parameters

Rank-Order Correlation	Comp MC %	DD (t/m^3)	OMC (%)	MDD (t/m^3)	CBR - 2.5 mm	CBR - 5 mm	Swell %	Avg MC after soak	DD (t/m^3) after soak	MR at comp	MR soaked	DR at Comp	DR soaked	e before	e after	Av before	Av after	DOS before	DOS after
Comp MC %	1.00																		
DD (t/m^3)	-0.84	1.00																	
OMC (%)	0.84	-0.89	1.00																
MDD (t/m^3)	-0.88	0.97	-0.92	1.00															
CBR - 2.5 mm	-0.47	0.61	-0.49	0.59	1.00														
CBR - 5 mm	-0.56	0.66	-0.49	0.66	0.90	1.00													
Swell (%)	0.41	-0.62	0.57	-0.64	-0.79	-0.75	1.00												
Avg MC after soak	0.43	-0.84	0.72	-0.82	-0.61	-0.65	0.66	1.00											
DD (t/m^3) after soak	0.39	0.20	-0.04	0.22	0.53	0.52	-0.58	-0.68	1.00										
MR at comp	0.42	-0.05	-0.07	-0.06	0.03	-0.08	-0.34	-0.27	0.48	1.00									
MR soaked	-0.02	-0.19	-0.11	-0.11	-0.76	-0.73	0.68	0.50	-0.53	0.06	1.00								
DR at Comp	0.01	0.27	-0.04	0.05	0.18	0.11	-0.05	-0.29	0.14	0.10	-0.14	1.00							
DR soaked	0.64	-0.17	0.36	-0.28	0.56	0.55	-0.59	-0.36	0.85	0.53	-0.61	0.13	1.00						
e before	0.74	-1.00	0.89	-0.92	-0.38	-0.45	0.45	0.84	-0.20	-0.03	0.19	-0.44	0.17	1.00					
e after	-0.39	-0.20	0.04	-0.22	-0.53	-0.52	0.58	0.68	-1.00	-0.48	0.53	-0.14	-0.85	0.20	1.00				
Av before	-0.69	0.16	-0.22	0.15	-0.55	-0.52	0.50	0.16	-0.52	-0.76	0.34	0.05	-0.61	-0.16	0.52	1.00			
Av after	-0.59	0.37	-0.36	0.37	-0.31	-0.33	0.43	-0.12	-0.58	-0.48	0.20	0.03	-0.79	-0.37	0.58	0.52	1.00		
DOS before	0.79	-0.27	0.33	-0.27	0.45	0.41	-0.39	-0.04	0.51	0.76	-0.26	-0.07	0.63	0.27	-0.51	-0.98	-0.52	1.00	
DOS after	0.58	-0.40	0.37	-0.40	0.28	0.30	-0.39	0.16	0.55	0.46	-0.17	-0.05	0.77	0.40	-0.55	-0.50	-1.00	0.51	1.00

Table 10: Comparing correlation matrix ranking between parameters

Parameter	Linear Correlation (Pearson)			Spearman Rank Order		
	Rank	CBR-2.5mm	CBR-5mm	Rank	CBR-2.5mm	CBR-5mm
CBR – 2.5mm	1	1.00	0.97	1	1.00	0.90
CBR – 5.0mm	2	0.97	1.00	2	0.90	1.00
Swell (%)	11	-0.46	-0.49	3	-0.79	-0.75
MR soaked	13	-0.39	-0.40	4	-0.76	-0.73
DD (t/m³)	5	0.63	0.60	5	0.61	0.66
MDD (t/m³)	7	0.51	0.58	6	0.61	0.66
OMC (%)	10	-0.46	-0.50	7	0.61	0.66
Avg MC after soak	3	-0.69	-0.74	8	-0.61	-0.65
DR soaked	6	0.52	0.51	9	0.56	0.55
A_v before	18	0.03	-0.01	10	-0.55	-0.52
e after	9	-0.47	-0.46	11	-0.53	-0.52
DD after soak	12	0.46	0.45	12	0.53	0.52
Comp MC %	8	-0.49	-0.54	13	-0.47	-0.56
DOS before	14	-0.24	-0.19	14	0.45	0.41
e before	4	-0.66	-0.71	15	-0.38	-0.45
A_v after	16	-0.17	-0.18	16	-0.31	-0.33
DOS after	17	0.13	0.14	17	0.28	0.30
DR at Comp	19	-0.01	-0.08	18	0.18	0.11
MR at comp	15	-0.17	-0.21	19	0.03	-0.08

In conclusion, there is a poor relationship with CBR and density ratio. A higher strength / modulus is associated with a higher DR, but one should not apply the reverse logic and associate a high DR with a high strength or modulus. Yet most practicing engineers (**Table 1**) incorrectly believe that the maximum CBR occurs at the MDD (100% DR).

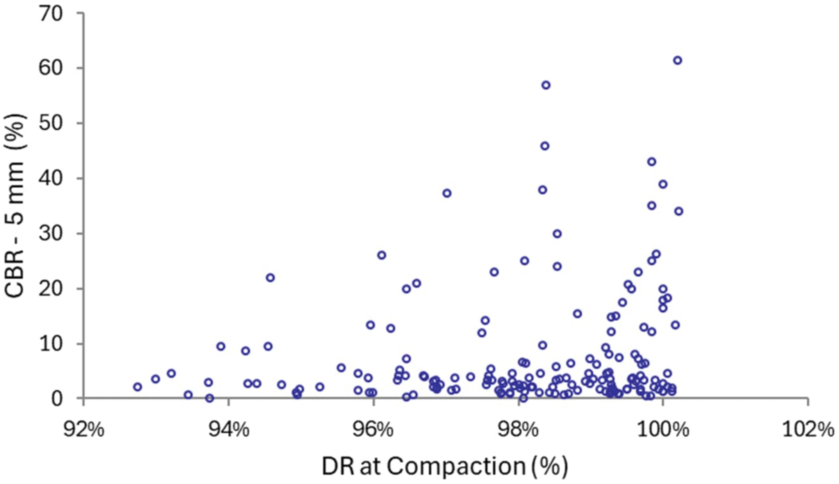


Figure 30: CBR is not dependent on the density ratio (n= 172)

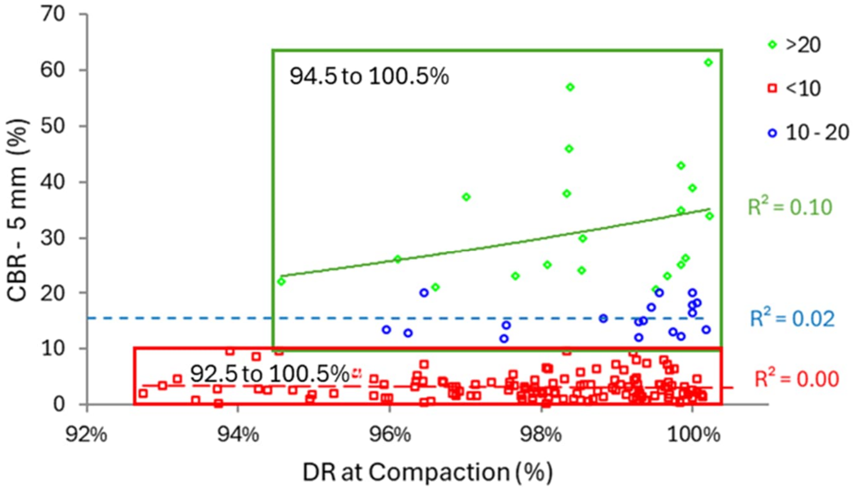


Figure 31: High CBR requires a high DR. This should not be extrapolated as higher DR produces a higher CBR

The following moisture ratios are required to be achieved (**Figure 32**) to ensure the corresponding CBR is achieved:

- CBR > 20% : MR = 75% to 105%
- CBR = 10 to 20% : MR = 70% to 115%
- CBR < 10% : MR = 65% to 130%

The compaction moisture range should be related to the CBR and EMC and not to the OMC. The latter represents a correspondent value related to achieve the MDD but has risen above its importance, as if it were also a target value.

These DR and MR graphs show that a high strength material (CBR > 20 %) requires a narrow range. This validates our state of practice. However, the same logic is then forced on to lower strength material as if high compaction of a CBR < 10% material can achieve high strength. It does not. In such materials a wide range of DR and MR can achieve the same strength.

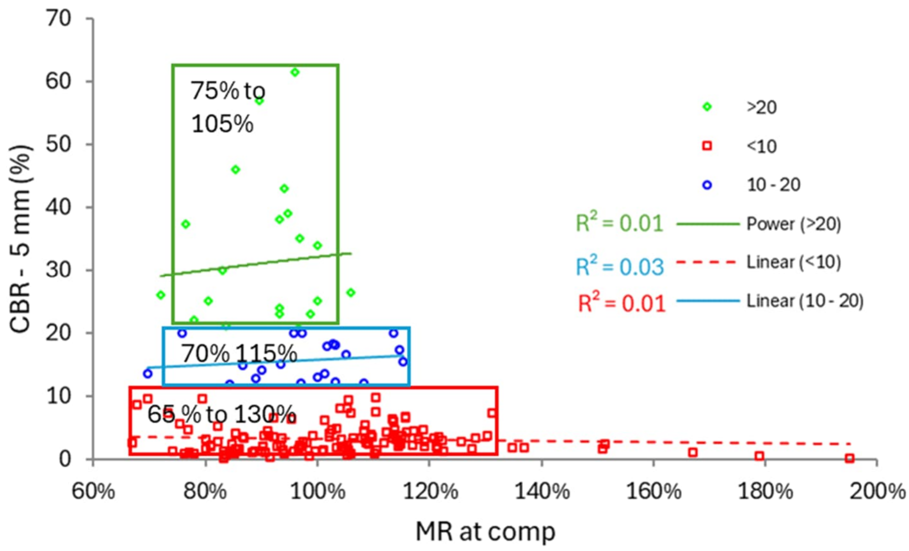


Figure 32: Higher CBR material requires a tighter moisture ratio (MR) at compaction (n= 172)

5. CONCLUSIONS AND RECOMMENDATIONS

There are many inconstancies in testing, with industry practice not consistent with what is considered a trusted or not trusted test. This is partly due to large testing variation in most tests. The density ratio and CBR are ubiquitous in CMT and were used to illustrate the many variables assumed in such testing laboratory models. In particular, consideration of curing time and oversize particles are required by testing standards and when these considerations are not rigorously applied, this may lead to significant variation in test values as demonstrated with case studies.

Laboratory and field tests are models. Understanding the limitations and application of such models are required. Design optimisation requires an appropriate test model and specifying:

- Equilibrium moisture content
- Soaked / Unsoaked CBR testing
- 4 day / 10 day soaking in applying the soaked CBR test
- Surcharge value
- Avoiding 1 point CBR testing in variable and / or high swell materials

It should not be left to the laboratory to specify the above as default values. The lab model relating MDD and OMC are useful construction indices, but a lab model does not consider the related EMC or CBR which are more relevant to long term conditions.

Case study data was compared using advanced statistical techniques to show:

- Use of a trend line automatically generated or normal PDFs has associated bias with outliers included
  - Not using the default Pearson Coefficient (as used in EXCEL) for high outliers or non-normal PDFs.
  - Comparing correlation matrix rankings between Pearson and Spearman correlations was shown with CMT data
- In multivariate analysis, using other techniques such as dendrogram analysis. This was compared with results in the standard bi-variate graphs and tabulated correlation matrices. This can be intimidating with data overload and time consuming to carry out.

The high precision of density testing hides its inaccuracy to assess key design parameters at high compaction levels. Once a minimum DR of say 92% is achieved to reduce the air voids, any increased compaction as judged by a DR value has little relation with strength or modulus. Both DR and MR have construction ranges related to the type of material. Since high strength materials can achieve a high DR, the reverse logic has incorrectly being widely applied, i.e. a higher DR achieves a higher strength. Assuming a DR of 98% is stronger than a DR of 95% is not necessarily correct and other testing is then required. Correlation errors occur when relating DR to more accurate tests.

For a high strength material (say CBR > 20%), compacting to a high DR (say 98%) will have a benefit.

Testing variation combined with broad correlations suggests our placid acceptance of errors simply because as an industry we have grown to accept such historical inaccuracies. A few key takeaways with testing are:

- Density is not performance. At high values density is poorly correlated to strength or modulus (stiffness).
- Strength is not linearly related to stiffness
- Modulus is not a constant

In the case of working platforms, designs have minimum modulus or strength requirements, and density testing alone is insufficient. Case studies on such applications are provided by Look and Honeyfield (2016), Barounis and Smith (2017), Barounis and Philpot (2025).

This paper focused on local Australia and New Zealand CMT practices and data, but the conclusions are not unique. Riad at al. (2023) show that density based quality control specifications do not provide engineering properties that can be used to ensure optimal performance of tested unbound pavement materials. Modulus based specifications have been successfully used by many USA state authorities and European countries. However greater spatial test data variability and the significant effect of moisture content are some of the challenges. This is no different from the early work of Selig and Truesdale (1967) shown in **Table 3** when precision took precedence over accuracy in elevating DR testing to its current prominence. Referring to that early and more comprehensive research by Selig and Truesdale (1967) they found the order of importance as:

1. Moisture was **by far** the most significant factor influencing the measurements. The MR soaked accounts for 73% to 76% of variance of CBR according to the Spearman Rank Order (**Table 10**) and was the highest influence
2. Soil type
3. Compaction equipment. Effectiveness depends on soil type. Effect changed with soil thickness
4. Lift thickness. Moisture and thickness had little interaction. Significant effect on density
5. Compactive effort. The DR at compaction accounted for 11% to 18% of contribution to the soaked CBR value (**Table 10**)
6. Dry density measurement was the least variable while field CBR, Moisture content and plate load were the most variable

This paper did not have cover all of the important factors above. Permeability is also an important factor but no data was available for such discussion.

The primary objective of compaction is to reduce the air voids and current practice achieves this indirectly by the Proctor MDD and OMC approach. CMT then often use the MDD and OMC as an aim, but this is incorrect. It is a simple and precise test to infer air voids (to reduce settlement) has been reduced to an acceptable level. With currently available technology and this research of influence of test variables, the recommendation is for a 3 step quality process

1. **Step 1**: Use the conventional DR approach to reduce air voids and control settlement. Once compaction to over 90% is achieved the air voids is likely 5% to 10% air voids. But density is not a performance indicator. As shown high density is poorly related to strength or modulus
2. **Step 2**: Assess strength or modulus. Many testing tools are available but such discussion was not the focus of this paper. These include plate load test (PLT), Clegg impact hammer, Geogauge, geophysical based devices and Light falling weight deflectometers (LFWD). A challenge is that the "modulus" values can vary between these different equipment due to varying strain levels and depths of influence. Different commercially available LFWD equipment may measure different values at the same site. A key consideration is not to correlate to DR due to unreliable correlations (Look, 2019).
3. **Step 3**: Assess uniformity of site compaction with a representative quantity of testing now economically possible with some of the testing equipment mentioned above. Proof rolling has traditionally been used in parallel with DR, but this "proof roll test" also has its limitations. Intelligent compaction would better serve industry in this regard.

CRediT authorship contribution statement

Burt Look: Writing - original draft.

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APPENDIX – RELATIONSHIPS OF DENSITY, STRENGTH AND MODULUS

We often default to density as if it is a proxy for performance. At the macro level, strength increases with density was shown in **Figure 9**. That data is now repeated with the correlation (R^2) of 0.65 and 0.61 for the high and low modulus values, respectively (**Figure 33**). This may seem a reasonable correlation. Removing the data points for man-made materials (high strength and low strength steel and concrete with its density and modulus values) result in the correlation (R^2) of 0.23 and 0.05 for the high and low modulus values, respectively (**Figure 34**). This is now a weak correlation. This shows modulus is not related to density.

Using 131 undrained strength results in residual soils (**Figure 35**) also show no correlation between strength and dry density correlation (R^2 = 0.04).

These results suggest the common assumption of strength or modulus is related to density is incorrect. Yet density testing is the basis of quality control in CMT and we then assume appropriate strength or modulus for that material.

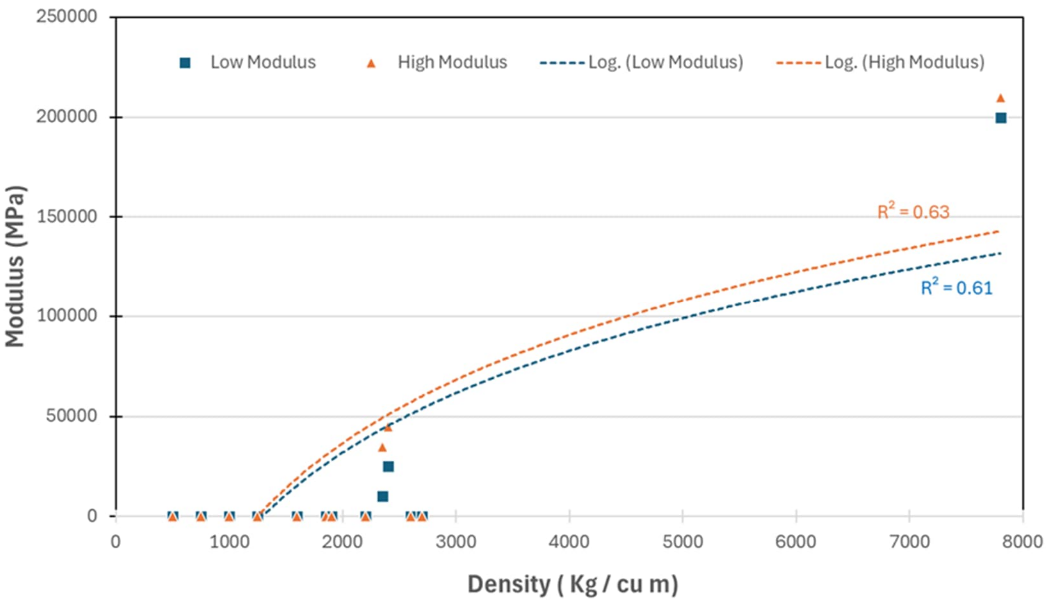

Figure 33: Change in modulus with density for varying material

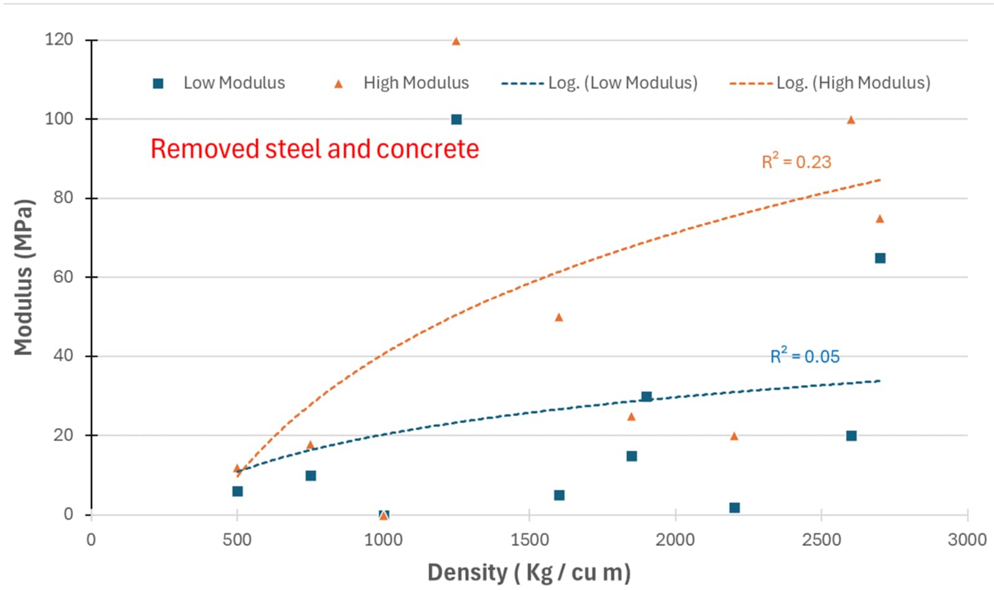

Figure 34: Change in modulus with density for material with steel and concrete values removed

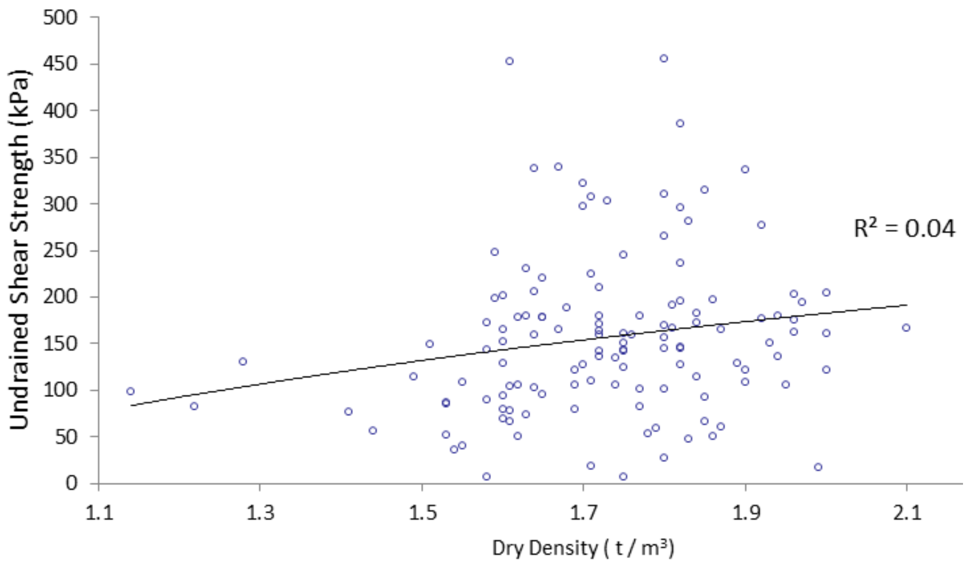

Figure 35: Change in undrained strength with dry density for residuals soils

A SYMMETRICAL TALE

Philip Pells, Tony Barry and Neil Fimeri
Semi-Retired

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ABSTRACT

The Sydney Opera House Carpark is now 35 years old. This is part of the untold story, a significant milestone in Australian Geomechanics

1. INTRODUCTION

The Sydney Opera house is widely acknowledged as one of the world's great buildings. Its beauty is correctly attributed to the Danish architect Jorn Utzon.

Many engineers, and a reasonable proportion of the Australian public, are aware that Utzon's concept for the roof shells was a structural impossibility. It took the technical and persuasive skills of Ove Arup, and his senior engineers, Mike Lewis and Jack Zunz, to create a structurally sound rib system that substantially retained Utzon's roof shapes.

The recent biography by Peter Jones sets out the story of this major contribution of Ove Arup to the structure of the Opera House. It also notes the pivotal role played by this famous project in the development of the Arup firm.

Buried within the biography is a gem of information, unknown to all those involved in subsequent developments at the Opera House site. This information adds a wonderful denouement to the association of Ove Arup with the building. This is the first telling of the full true story.

2. AN OPERA HOUSE WITH NOWHERE TO PARK

When the Opera House opened in 1973 for its first performance, Prokofiev's *War and Peace*, there was no nearby place for parking. Patrons then, and two million annual patrons that followed over the next 20 years, had to walk from the train and ferry terminals at Circular Quay or find parking places in small business parking stations in the CBD.

Many schemes for parking stations to serve the Opera House were proposed as part of its original design, and over the following years. However, for financial and environmental reasons these came to nought.

3. THE CARPARK HAPPENS

After various false starts over more than 15 years, the New South Wales Government enacted the Bennelong Point Parking Station Act (1985) and put out a tender in early 1990 for private enterprise to build and operate a 900-vehicle underground car park to service the Opera House.

The Public Works Department provided a four-cavern reference design which had been developed by Arup and used as the basis of the Environmental Impact Statement by Planning Workshop in 1988. The facility was required to be built in a restricted footprint area beneath the Botanic Gardens, only a few hundred metres from the Opera House forecourt.

The Mulpha Group (Enacon Parking Pty Ltd) submitted a conforming tender that comprised two side-by-side rectangular caverns with cross connections at the ends. Each rectangular structure was much like a traditional aboveground parking station, with narrow ramps and tight corners. And if you were the last patron in either cavern you would have to wind your way down seven or eight levels and occupy the last place in the bottom corner.

The design had other unattractive features. Ventilation of the two chambers was difficult and expensive, and egress stairs knocked out a significant number of valuable parking spaces. It didn't even fit into the allocated parcel of land. This was exacerbated when the Public Works Department asked during negotiations for an additional 200 spaces for Opera House employees to be accommodated in the car park. This was very costly and technically challenging.

Much to the surprise of most of the team members including the first two authors of this article, the Government accepted Mulpha’s submission. Basically, the competitors’ designs were much the same, and Mulpha had a better financial deal.

The preferred tenderer nomination was made in February 1990 on a Friday and most of the team members went home. In the days following, the Rankine & Hill team became concerned about the compromises evident in the reference and tender designs. These were drawn up and mapped out.

Messrs Barry, Colefax, Reid and Ferguson from Rankine & Hill met with architect Ron Barelle on the following Wednesday to discuss the issues of the cavern footprint, ventilation shafts, lost parking places to accommodate fire escapes, and having to drive down tight corners into the bowels of the earth to get to the last parking place,

Towards the end of the meeting Ron Barelle was sitting opposite Warwick Colefax, whose hands were clasped around a beer can. Ron said he had seen an aboveground parking station in Paris that was a helix, and it self-ventilated up through the spiral. Being an engineer who had learned to be patient with architects, Warwick pointed out that this would not work underground. But then, probably encouraged by the contents and shape of a beer can, the lights started going on and they started to sketch.

Within an hour they called Neil Fimeri, Mulpha’s Project Director. Tony Barry, Warwick Colefax and Ron Barrelle, met in the Union Hotel that Wednesday evening and continued their discussions of issues with the tender design.

They described the potential for an annular cavern footprint with a double helix spiral carpark to solve most of the issues. Neil authorised them to immediately work up an alternate design.

The Rankine & Hill team developed the initial concept of a 30m deep cavern, circular in plan with a central rock pillar, and this donut shaped cavern contained a double helix concrete structure that could be interconnected at any location simply by tunnelling horizontally through the central pillar of rock. Meetings with the architect and all engineering disciplines were scheduled for Thursday to make sure all the needs of various disciplines could be accommodated.

The gently sloping ramps of the helix provided for parking and access. Now the last parking place was not at the bottom, but at the top. Ventilation was easy with plant at the top level and vertical shafts around the circumference of the inner and outer edges of the cavern; only one cavern. Travel distances were substantially reduced and hence the required number of fire stairs was also reduced. The total volume of excavation was reduced, and low and behold, it could be fitted within the originally allocated development boundaries. The footprint area was reduced from 7900 square metres to 3000 square metres. The concept drawings were developed by Friday evening and sent to Neil Fimeri and Philip Pells at Coffey and Partners.

A stringent condition of tender was that under no circumstances was the ground surface in the Botanic Gardens to be disturbed for any purpose, and so the key question became: Could such a cavern be built, with only 6m of rock cover?

On the following Monday the first author, who was then a Principal of Coffey and Partners, turned up at work, oblivious of Wednesday’s beer befuddled discussion, and the detailed work that followed over the weekend. Mid-morning a fax arrived followed by a phone call. Simple question: Could such a cavern be built? I had never seen anything like it. Nobody had, because it had never been done before.

If one is very fortunate then perhaps once in a lifetime one will see a concept or design of sheer brilliance. That’s what appeared that morning. Furthermore, the requirement for an 18m span cavern with only 6m of Hawkesbury Sandstone cover kicked in an idea I had been mulling over ever since Dick Bieniawski pointed me to a 1940’s paper by Evans on linear arches. After some calculations of the amount of rock reinforcement necessary to create the requisite linear arch, it was concluded that the cavern could be built with no disturbance of the ground surface. The internal concrete helix structure does not support the sidewalls or crown of the cavern.

The team met again on Tuesday to work through how to present this to the Public Works Department and in doing so realised that it would be possible with a small amount of additional excavation to deepen the cavern to 36 metres which would provide the additional 200 car spaces for Opera House employees, from the savings achieved with the circular cavern design. The Rankine & Hill team went to work amending their drawings and Ron Barelle begun preparing architectural drawings for presentation to the government. Neil approached Glenn Monckton, the Public Works Department Project Director and with the team presented the alternate concept design. Soon after the team was invited to meet with Minister Wal Murray on the following Monday morning at 6.30am.

Over the weekend, architectural and engineering concept drawings were finalised with a small cardboard model of the double helix car park, with cross tunnels and car park markings and fire stairs to assist the team to understand how it would work.

By the Wednesday of that week the NSW Government agreed that the drawings upon which they had awarded the project could be thrown away, and the double helix was adopted. Enacon was given 60 days to lodge a development application for the alternate design.

However, in the background there was a problem. As indicated in Figure 1, the ground surface slopes gently down towards Sydney Harbour. With the proposed design, drivers on the exit (upwards) spiral were moving clockwise; and with the exit tunnel level being constrained by the Sydney Harbour Tunnel, I, the first author, concluded there simply was not sufficient rock cover on the harbour-side of the tunnel. Reluctantly I communicated this to Tony Barry, the second author. There was silence on the phone for a short while, and then he said. “No problem, we will just reverse the direction of the spirals”. That gave enough rock cover and that is why drivers exiting the carpark are travelling anticlockwise.

Three years later, on 17th March 1993, the facility was opened. It works brilliantly, even if some patrons get confused by the double helix.

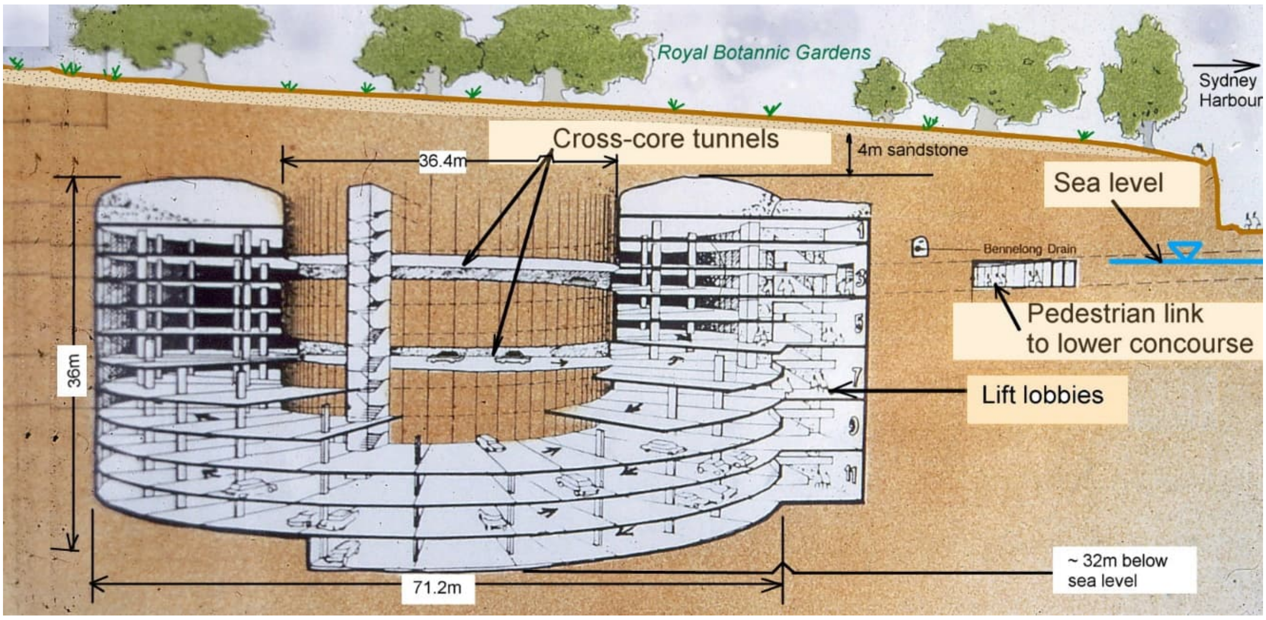

Figure 1: Artist’s sketch of the double helix structure

Figure 1 is the original artist’s sketch of the underground facility. Figure 2 shows how it fits in plan.

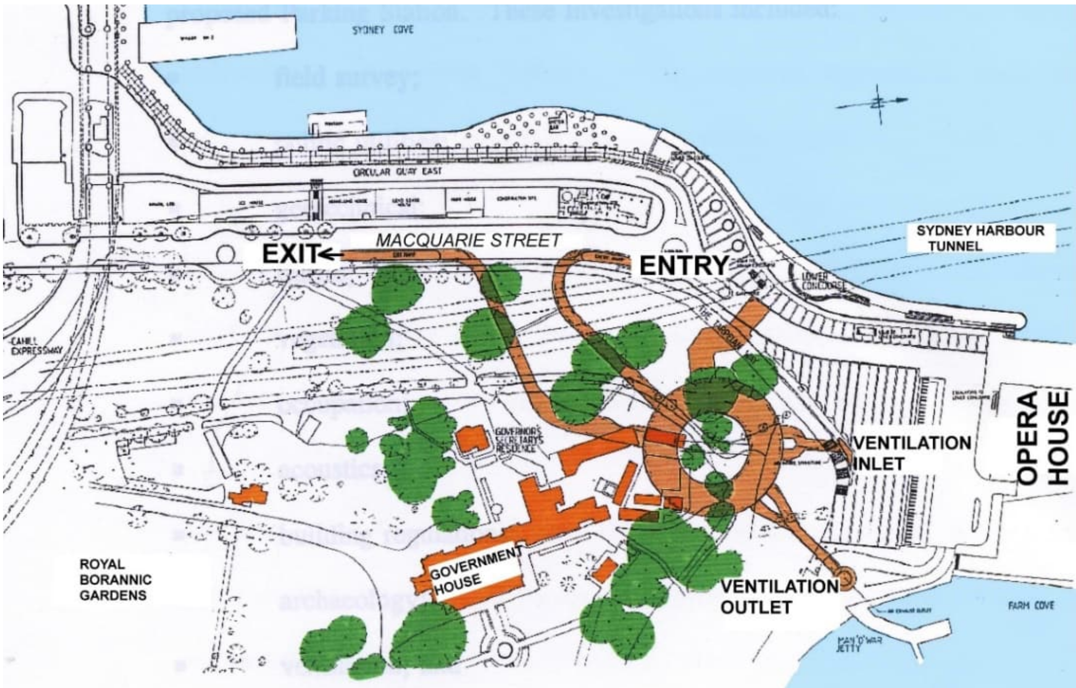

Figure 2: How it all fits in plan.

The carpark comprises a freestanding 12-storey double helix structure within the 140,000 cubic meter cavern. The cavern is about 60 meters from the shoreline of Sydney Harbour, and the floor is about 34 metres below sea level.

Figures 3 to 7 show parts of the cavern, during construction; the whole of which defied photography.

Figure 3. Excavating the crown of the cavern using Mitsui roadheader

Figure 4: Commencement of bulk excavation of Hawkesbury Sandstone using impact ripping

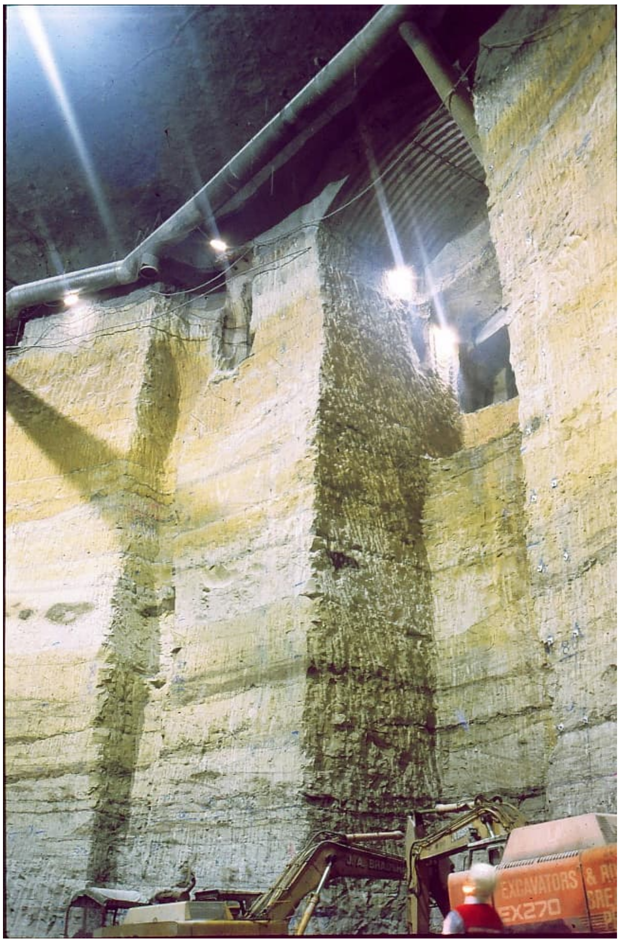

Figure 5: Near the base of the excavation, showing the sidewall slot that contains the lifts and stairs

Figure 6: Cross-core tunnels

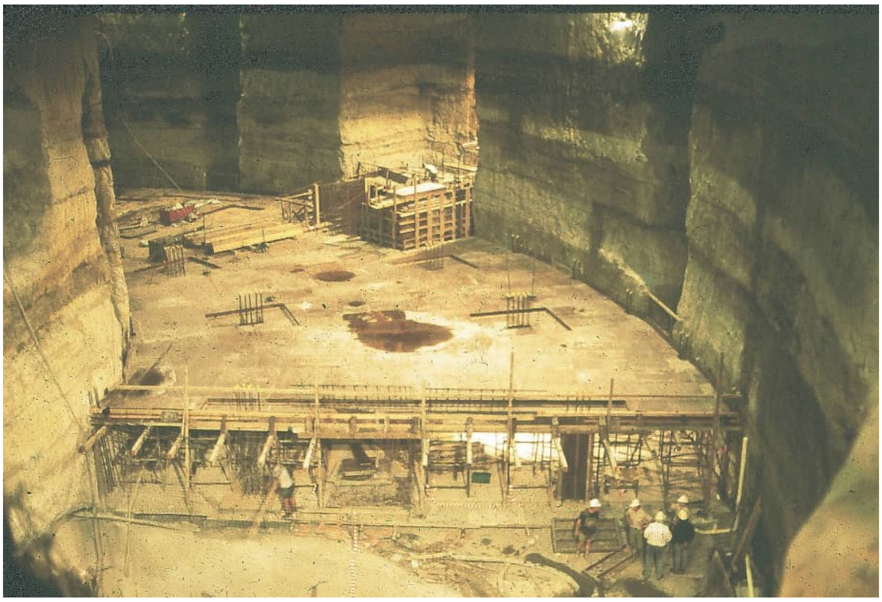

Figure 7: Commencement of the concrete double helix; stairs and lift structure being built in the slot

4. WHO FIRST THOUGHT OF A DOUBLE HELIX CARPARK?

Fascinated by the elegance of the double helix design, I, the first author, spent quite some time trying to find out who first thought of using a double helix in a building of any sort. This quest took me to fascinating places, from Chambord Castle on the Loire, a Leonardo da Vinci double helix staircase, to the double helix well in Orvieto, (*the Il Pozzo della Rocca di San Patrizio)*, and to thought meetings with amazing people, from Archimedes to the Iranian architect of the Minaret of Jam in Afghanistan. But throughout this process I, like all others, involved in the project, had no doubt that the Sydney Opera House facility was the first time a double helix structure had been designed for an underground car park.

It was with this certainty that in March 2007, on a visit to my son Steven Pells in Cairns, I removed the plastic wrapper from the Peter Jones biography of Ove Arup that had been given to all staff members of Arups, all 5500 of them. In the steamy heat of Cairns, I browsed vaguely through the section describing Ove Arup's involvement in the Opera House and then couldn't believe my eyes when I saw Photograph No 32, reproduced herein as Figure 8. What it showed was that in 1938, as part of designing air raid shelters for London, Ove Arup had designed an underground helical structure that was intended to, after hostilities, act as a car park. Well, I thought, at least he didn't think of a double helix. But I had only to turn to page 74 to find out that I was wrong. As Peter Jones says: *"In his March version, Ove designed ramps in the form of two helixes, one inside the other, echoing his design of the penguin pool at London Zoo."* Unfortunately, we have been unable to locate the drawings of the double helix version, but we accept Peter Jones' statement.

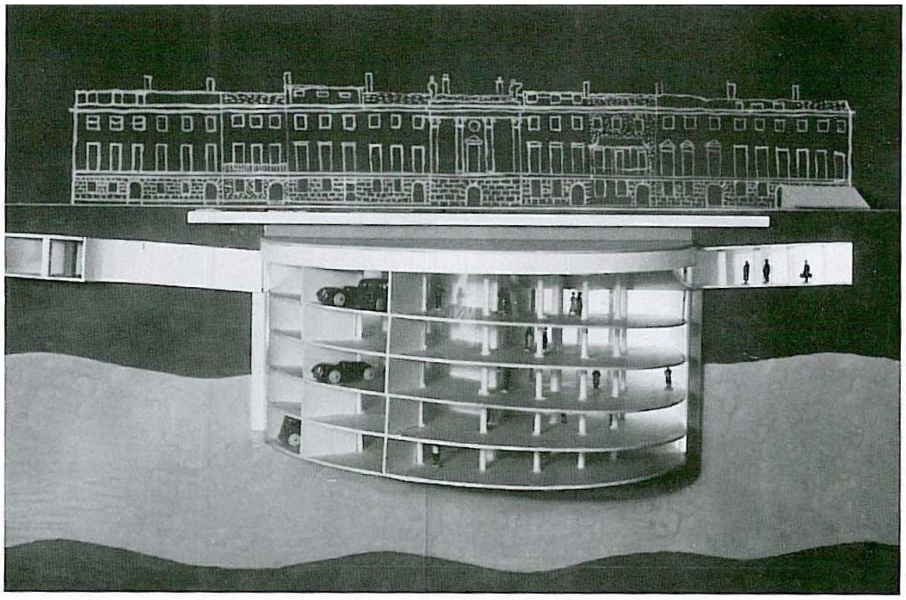

Figure 8: Ove's design for a helical underground shelter in Finsbury, intended for later use as a carpark (1938)

So, incredible as it seems, the double helix structure of the car park of the Sydney Opera House, was created unknowingly by Messrs Colefax, Barry and Barrelle to almost exactly mimic a 55-year-old, never-built, Ove Arup design. We can only wonder:

- "Would Ove Arup, personally, have come up with the double helix design for the car park if he had been retained by one of the groups that submitted turnkey designs in response to the NSW Government tender of 1990?"

5. DENOUMENT

Tony Barry's little story

In February 1993, Tony documented the development of the Opera House Car Park for his Master of Engineering Science thesis "The Bennelong Point Parking Station".

The late, and great, Professor John Booker at Sydney University reviewed the document and asked if Tony wanted it considered for a PhD given the substantial nature of the work; he agreed.

Some months later Tony was called in to meet John, who indicated the thesis had been sent to five examiners, four of whom agreed to consider it worthy of a PhD but one, from Arup, in London had suggested the idea was not innovative as Ove Arup had used it in the design of air raid shelters in 1938.

Tony was awarded his Master of Engineering Science Degree.

The heroes from the rock mechanics viewpoint

Turning the concept of a linear arch, only 4m to 6m thick, into reality was the result of exceptional work.

Firstly, Prof Harry Poulos turned his knowledge of laterally loaded piles upside down and at a considerably reduced diameter, to model the resistance of fully bonded rockbolts under bedding plane shear. Then Prof John Booker conjured up the mathematics of a linear arch as a 1-dimensional finite element system and scribbled the equations on a piece of paper. And then Ross Best took John's equations and coded them for the minicomputer at Coffey's North Ryde office.

Then Prof Ted Brown, as external reviewer, added long shear reinforcement anchors at the haunches.

This work was documented in 1990, before the cavern was excavated (Pells , Poulos, Best, 1990).

Authors; 35 years ago

Neil Fimeri was the Project Director for Mulpha Australia and was the man who made it all happen, from concept, through design, on to construction and then operation as a car park. Being Neil, he has now morphed into a mechanical engineer manufacturing large scale air-conditioning facilities.

Tony Barry was with Rankine and Hill and together with Warwick Colefax and others took a mad design concept and made it work. He went on to be CEO of Aurecon and along the way was awarded an AM.

Philip Pells was a Director of the then private company Coffey & Partners. He was given the flick pass of designing and implementing the rock reinforcement system for what might be still the widest low cover rock cavern in the world; although years later having visited the extraordinary 2000-year-old caverns at Longyou in China maybe they take the crown. He went on the help establish the firm Pells Sullivan Meynink, now known just as PSM

CRediT authorship contribution statement

Philip Pells: Writing - original draft. **Tony Barry:** Writing – review and editing. **Neil Fimeri:** Writing – review and editing.

5 REFERENCES

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Pells, P.J.N., Poulos, H.G. and Best R.J. (1990). Rock reinforcement design for a shallow large-span cavern. *Proc, 7th Int. Conf. Rock Mechanics*, Aachen.


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EMBEDDED RETAINING WALL DESIGN IN ACCORDANCE WITH AUSTRALIAN DESIGN STANDARD AS5100.3-2017

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ABSTRACT

Australian Standard AS5100 is often specified as the technical standard for embedded retaining walls in Australian infrastructure projects. The authors have identified compatibility issues between strength and stability-based criteria in AS5100 and Strength Factor analysis methods adopted by commonly used software packages. AS5100 adopts a Limit State Design approach requiring a reduction factor to be applied to passive restorative forces; whereas the Strength Factor method used in commonly adopted software such as WALLAP (Strength Factor Method option) and PLAXIS, applies a single reduction factor to all values of soil strength in the active and passive zones. The fundamental difference creates challenges for designers attempting to demonstrate compliance with AS5100 when using the widely adopted software packages.

Using a typical cantilever wall example and a range of commonly adopted soil parameters, the authors demonstrate equivalency between these methods by ascertaining the minimum Strength Factor required to achieve equal (or longer) embedment depths compared with the stipulated reduction factor per AS5100.

1 BACKGROUND

Embedded retaining walls are a common type of retaining structure to support excavations in soil and rock. Adherence to Australian Standard AS5100 – Bridge Design is commonly stipulated in the technical specifications for the design of retaining walls in Australian infrastructure projects.

The authors have identified challenges faced by designers to prove the attainment of strength and stability-based criteria in AS5100, due to the inherent incompatibility with analysis methods adopted by commonly used software packages such as WALLAP (Strength Factor Method) and PLAXIS. In this context, strength and stability-based assessments pertain to the embedded length of the pile, i.e. the length of the pile below the excavated ground surface.

For geotechnical strength (failure) and / or stability-based assessments AS5100 adopts a Limit State Design approach, requiring reduction in the restoring forces and increase of applied forces with load factors. Commonly used design software such as WALLAP (Strength Factor Method), PLAXIS2D and PLAXIS3D employ the Strength Factor method which progressively apply a single reduction factor to all values of soil strength in both the active and passive zones until a state of failure or instability is reached. The calculation methods of AS5100 and common design software are seemingly incompatible, i.e. FOS (factors of safety) calculated from WALLAP (Strength Factor Method) and / or PLAXIS are not equivalent to the FOS (inverse of load factors) stipulated in AS5100.

The incompatibility between AS5100 and design software has created inefficiencies in practice. Many designers have to perform duplicate calculations to prove that the embedded wall designs are satisfactory using their adopted design software and then verifying compliance with AS5100 Limit State Design approach. This has resulted in design inefficiencies and re-work in tight working schedules.

This paper aims to review the requirements of AS5100 and ascertain design “equivalent FOS” that are compatible with current modelling software / approaches. Note that this paper focuses on geotechnical strength design only. Structural design and serviceability should also be considered but are not included in this paper.

2 WHAT IS AN EMBEDDED RETAINING WALL

An embedded retaining wall is one that penetrates the ground at its base and obtains some lateral support from it (CIRIA C760). The wall may be freestanding or cantilever and may also be supported by structural members such as props, berms, ground anchors and slabs. Common retaining wall types include sheet piles, king post walls, contiguous bored pile wall, secant pile wall and diaphragm walls.

Cantilever embedded walls rely on an adequate embedment below the excavation level for their stability. For unpropped walls, the failure mechanism is likely to be rotational movement around the wall toe where the soil behind the wall will move to an active state while the resisting zone in front of the wall will provide passive support. Refer to Figure 1 for an illustration of the stress distributions around embedded walls extracted from CIRIA 760.

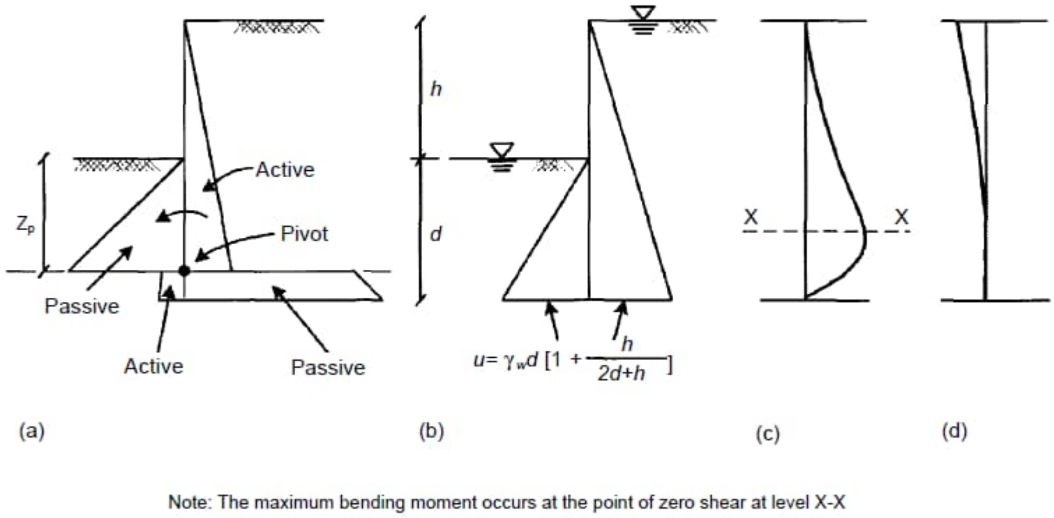


Figure 1: Idealised stress distribution for an unpropped embedded cantilever wall at failure: (a) effective stress; (b) pore water pressures; (c) wall bending moment distribution; (d) wall deflection (CIRIA 760)

3 METHODS OF ANALYSIS

3.1 OVERVIEW

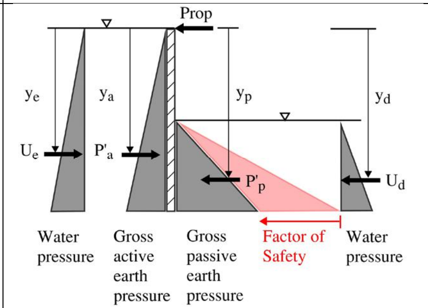
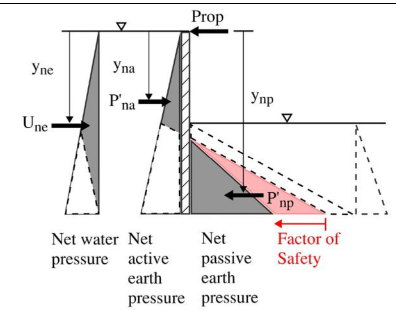
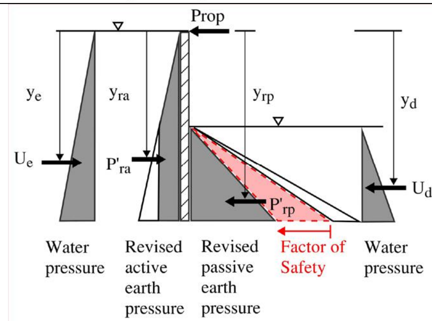
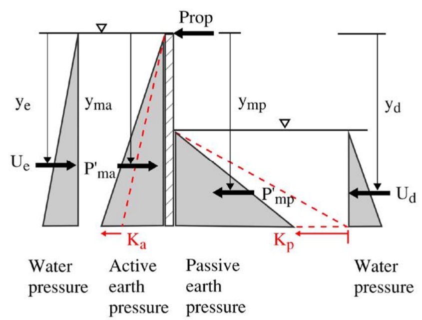
A brief history of embedded retaining wall stability design methods is summarised in Table 1. Some of these analysis methods are still in use today and form the basis for the geotechnical strength / stability assessments in commonly used design software.

4 OVERVIEW OF AS5100.3

The latest version of Australian Standard AS5100 Bridge Design was issued in March 2017, containing a total of nine separate parts. For the purposes of this paper, reference will be made mostly to Part 3 – Foundation and Soil-Supporting Structures (AS5100.3 – 2017) as this contains the design criteria for retaining walls. Reference will also be made to AS5100.3 Supplement 1 – 2008 which provides commentary and guidance to the application of AS5100.3 – 2004.

Note that an updated supplement is not currently available for AS5100.3 – 2017. The authors have undertaken a review of the design criteria in AS5100.3 – 2004 versus 2017 and identified no discernible differences. Hence, the authors consider use of AS5100.3 Supplement 1 – 2008 for the interpretation of AS5100.3 – 2017 remains adequate for the context of this paper.

Table 1: Embedded retaining wall analysis methods, based on Clayton, C.R.I et al (2013)

Method	Name	Description	Relevant Equations	Illustration	Comments and Notes
1	Civil Engineering Code of Practice No. 2, 1951 (CP2) (= AS5100.3)	• Published by Institution of Structural Engineers in 1951 • Apply single factor of safety (minimum 2.0) onto gross passive earth pressures	$FOS = \frac{P_p' y_p}{P_a' y_a + U_e y_e - U_d y_d}$		• Follows calculation methodology in AS5100.3 Supplement 1 – 2008, i.e. applies reduction factor to passive resistance only. Refer Section 5.5 • Burland et al. (1981) criticised the factors of safety on gross pressures when applied to undrained conditions as the method rendered inconsistent embedded lengths, i.e. longer embedment depths were required for lower factors of safety • Superseded by BS8002 in 1994, British Standards Institution published new code of practice for earth retaining structures to replace CP2 This stability assessment method is adopted in WALLAP (CP2 option).
2	BSC Piling Handbook	• British Steel Corporation published Piling Handbook in 1997 (blue book). • Apply factor of safety to net passive earth pressure	$FOS = \frac{P_{np}' y_{np}}{P_{na}' y_{na} + U_{ne} y_{ne}}$		Burland et al. (1981) and Potts and Burland (1983) found that the factor of safety using the BSC method was very low compared with other calculated methods. For these reasons, the net pressure method has fallen out of favour in modern times
3	Burland Potts Method (Revised Method)	• Published by Potts and Burland (1983) • Attempt to counteract undesirable features of gross and net pressure methods (Method 1 and 2 respectively) • Effective earth pressures on both sides of the wall are reduced by an equal amount (by ignoring the hatched areas) so that the active earth pressures below formation level remain constant	$FOS = \frac{P_{rp}' y_{rp}}{P_{ra}' y_{ra} + U_e y_e - U_d y_d}$		• More complicated than the other methods and has not been used widely in UK practice (and to the author's knowledge hardly at all outside the UK). • With the publication of BS 8002 and Eurocode 7, use of the revised method has waned
4	Strength Factor Method	• In 1994 BS8002 introduced Mobilisation Factor, M, which reduces soil strength (drained or undrained) in the active and passive regions to a "design value". This is analogous to a "factor of safety, FOS" • Active earth pressures are increased; passive pressures are decreased.	$design\ c' = \frac{c_r'}{M}$ $design\ \phi' = tan^{-1}\left[\frac{tan\ \phi_r'}{M}\right]$ $design\ c_u = \frac{c_{ur}}{M}$ Where, $M = FOS$		• Supersedes CP2 (Method 1) in accordance with BS8002 • BS8002:1994: M = 1.2 for effective stress, and 1.5 for total stress parameters – applied to representative peak strength of soil • Stability assessment methods adopted in the following software programs have Strength Factor method functionality: o WALLAP – Strength Factor Method o PLAXIS – Phi/C reduction (Safety method) o RocScience – Shear Strength Reduction (SSR) In WALLAP / PLAXIS / RocScience, the Strength Factor is employed whereby the strength parameters are reduced by a factor, and by which all values of cohesion and tan phi must be reduced to bring about a state of limiting equilibrium, i.e. failure.

5 DESIGN METHODOLOGY ACCORDING TO AS5100.3

5.1 OVERVIEW

AS5100.3 adopts a limit state design method, whereby reduced restorative forces must exceed magnified destabilising loads. All strength-based assessments for the pile length must adhere to Equation 2.3.3(2) in AS5100.3, reproduced below in Equation 1.

$$\phi_g R_{u.g} \geq E_d \qquad \textbf{(1)}$$

Where the terms of Equation 1 are presented in Table 2.

Table 2: Terms in Equation 1

Symbol	Description	Details	Reference in AS5100.3 – 2017
E_d	Design Action Effect	Design effect imposed by loading, e.g. soil, surcharge on slope of ground surface, compaction pressures	Cl 2.3.3(d), Cl3.3.3, Cl8.3.2
ϕ_g	Geotechnical Strength Reduction Factor		Table 8.3.1 (A)
$R_{u.g}$	Ultimate geotechnical strength	• Passive resistance only • Use unfactored values for material parameters	Cl2.3.3(d)

Details of each input into Equation 1 are described in the sections below.

5.2 DESIGN ACTION EFFECT, E_d

According to AS5100.3, for soil-supporting structures (i.e. where the loads are imposed predominantly from the soil) load factors on the Design Action Effect, E_d, shall be 1.0 for geotechnical strength design. Given that design effects include the imposed loads, this pertains to any destabilising load, including (but not limited to) active pressures imposed by the soil.

5.3 GEOTECHNICAL STRENGTH REDUCTION FACTOR, ϕ_g

The reduction in restorative forces is governed by the Geotechnical Strength Reduction Factor, ϕ_g. A summary of the ϕ_g values stipulated in AS5100.3 is provided in Table 3.

Table 3: Geotechnical Strength Reduction Factors, ϕ_g, for retaining walls and abutments. Extracted from AS5100.3 Table 8.3.1(A)

Bearing Failure	Overturning, sliding and global stability	
	Permanent structures	Temporary structures
0.30 – 0.45	0.45 – 0.55	0.45 – 0.70

** selection of value adopted for design should be based on the method of assessment of ultimate geotechnical strength pertaining to quality of ground investigation undertaken*

Assessment of strength / stability for an embedded retaining wall pertains to overturning failure, and or global stability failure. As such, the relevant ϕ_g according to AS5100.3 Table 8.3.1(A) should be as per Table 4 below. The FOS for these reduction factors can be calculated from the inverse of each value. These inversed values will be referred to as FOS_{AS5100} here forth.

Table 4: Geotechnical Strength Reduction Factors (ϕ_g) and corresponding FOS in accordance with AS5100.3

Design case	ϕ_g	**Inverse of ϕ_g (FOS$_{AS5100}$)**
Temporary structures	0.45 – 0.70	1.43 – 2.22
Permanent structures	0.45 – 0.55	1.82 – 2.22

5.4 ULTIMATE GEOTECHNICAL STRENGTH

According to Cl2.3.3(d) of AS5100.3, the ultimate geotechnical strength is equal to the passive resistance on the retaining wall, determined using unfactored material parameters. It is upon the passive resistance that the Geotechnical Strength Reduction Factor, ϕ_g, is applied.

5.5 AS5100.3 SUPPLEMENT 1 – 2008 EXAMPLE CALCULATIONS

Within the appendices of AS5100.3 Supplement 1 – 2008 are a set of example calculations for an anchored pile wall demonstrating the limit equilibrium approach and limit state analysis in accordance with AS5100.3. The geometry of the problem is shown in Figure 2.

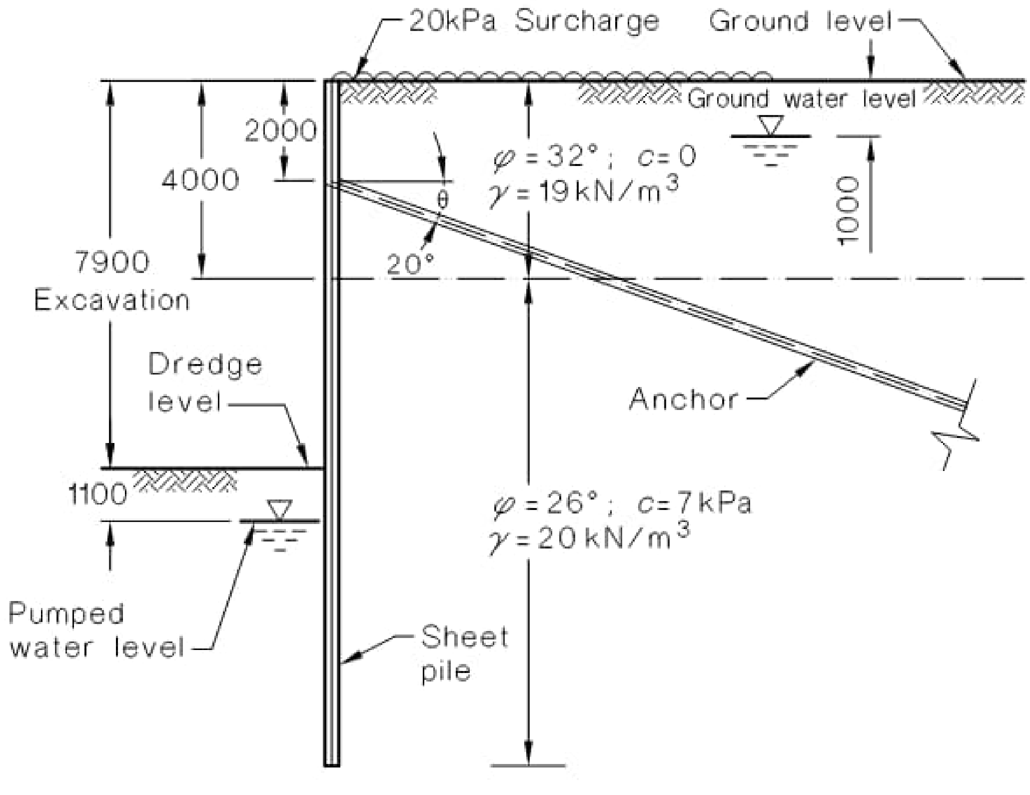


Figure 2: Embedded retaining wall example (extracted AS5100.3 Supplement 1 – 2008, millimetres)

The calculations summarised below have been extracted from the Limit Equilibrium (LEM) approach within the Supplement, which does not take into account interaction between pile, soil and flexibility of anchor and construction stages. Finite Element Modelling (FEM) and pseudo-FEM can take these details into account and are discussed in the Supplement. Reproduction of the LEM approach is used herein for transparency in the analytical application of AS5100.3.

In this example simplified distributions of active and passive pressures are assumed, using the nomenclature in Table 5.

Table 5: Nomenclature used in example extracted from AS5100.3 Supplement 1 – 2008. Refer to Figure 2 and Figure 3

Symbol	Description	Distance offset from anchor
$P_a^{'}$	Total effective active force	y_a
U_e	Total water load on active (retained) side	y_e
$P_p^{'}$ $(= R_{u.g})$	Total effective passive force	y_p
U_d	Total water load on passive (dredged) side	y_d

Taking moments about the anchor point, the depth of penetration needs to satisfy Equation 2. Note that in accordance with Cl2.3.3(d), the passive resistance should be factored by ϕ_g.

$$\phi_g\, P_p^{'} y_p \geq\; P_a^{'} y_a +\; U_e y_e - U_d y_d \qquad \textbf{(2)}$$

The pressures must be assessed and iterated for various pile depths until Equation 2 is satisfied.

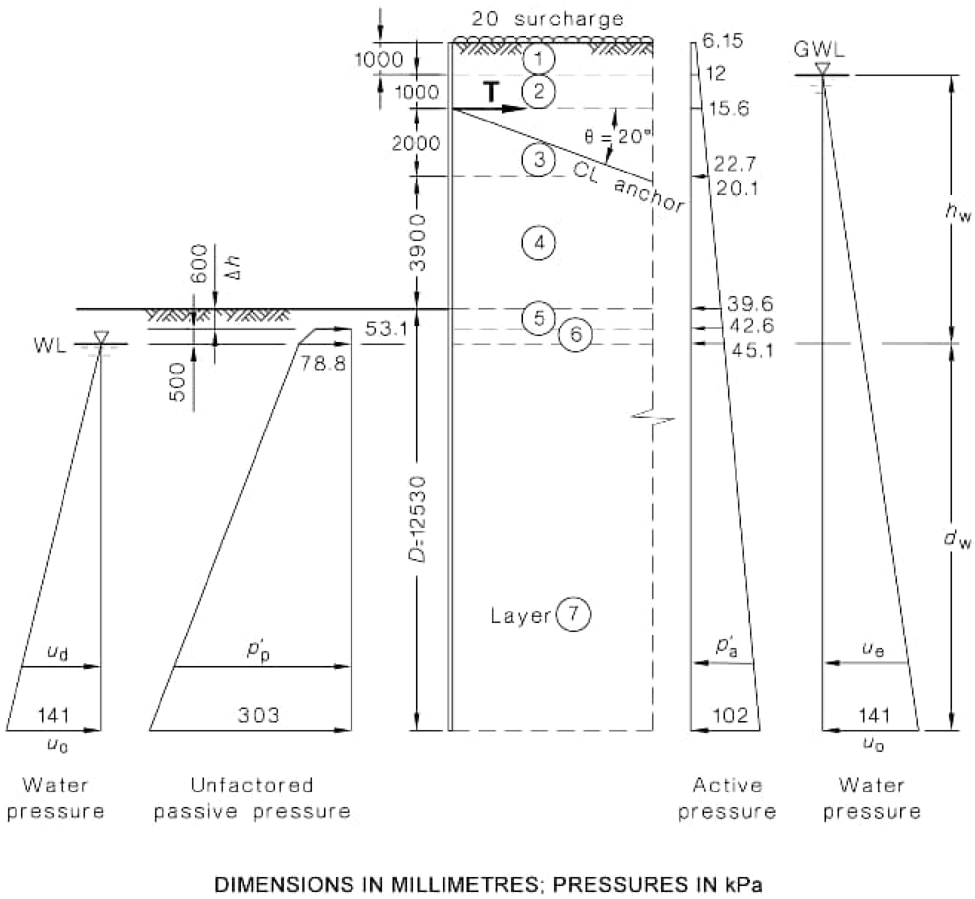


Figure 3: Embedded retaining wall example – calculated pressures (AS5100.3 Supplement 1 – 2008)

6 COMPARING ANALYSIS METHODS WITH AS5100.3 APPROACH

Of the analysis methods in Table 1, Method 1 (CP2) is equivalent to the LEM method outlined in AS5100.3 Supplement 1 – 2008. Their equivalency can be seen in equation for Method 1 from Table 1 versus Equation 2 from AS5100.3 summarised below. The equations are inverse of each other, i.e. FOS in Method 1 is relayed inversely as a reduction factor ϕ_g in Equation 2. The methods are identical in that the Strength Factor or Reduction Factor is applied to the gross passive resistance only.

$$FOS = \frac{P_p' y_p}{P_a' y_a + U_e y_e - U_d y_d}$$ Method 1 equation (CP2)

$$\frac{1}{FOS_{AS5100}} = \phi_g \geq \frac{P_a' y_a + U_e y_e - U_d y_d}{P_p' y_p}$$ Equation 2 (AS5100.3) rearranged

Methods 2, 3 and 4 adopt different means of applying the Strength / Reduction factors compared with the example in AS5100.3 Supplement 1 – 2008. As these analysis methods differ fundamentally from the AS5100.3 approach, results obtained using these methods do not demonstrate compliance with AS5100.3 criteria.

Method 4 (Strength Factor) is currently utilised within all commonly used software to assess FOS's for wall embedment depths such as WALLAP (Strength Factor Method), PLAXIS and RocScience. For wall designs using these (or similarly structured) software, engineers are required to use alternate means of demonstrating compliance with AS5100.3 criteria. This creates additional effort and cost in the design process.

7 SATISFYING AS5100.3 USING CURRENT ANALYSIS METHODS

7.1 GENERAL

Whilst the analysis methods within popular design software differs from that specified in AS5100.3, the sections below describe the method by which the authors derived "equivalent FOS's" using Strength Factor analyses (Method 4), i.e. FOS_{SF}, which match or exceed embedded pile lengths calculated using CP2 (ϕ_g, FOS_{AS5100}) per AS5100.3 (Method 1). Equivalency was not sought for Methods 2 and 3, as these are not commonly used design approaches.

7.2 ANALYSIS METHODOLOGY

Firstly, a list of the variables which were expected to have a significant impact on the geotechnical strength / stability of embedded walls was generated, including ranges of values that are typically encountered, as noted below. Refer to Figure 4 for an illustrated sketch of the noted variables.

- Soil strength, ϕ' (range = 24º to 38º)
- Retained height, H (range 2m to 7.5m)
- Surcharge pressure, q (range 0kPa to 50kPa)
- Water table height, W (range 0m to full retained height)

A parametric assessment was then undertaken where for each combination of the variables a geotechnical strength / stability assessment was performed using the CP2 method (using WALLAP CP2 option per AS5100.3) to establish the required embedment depth.

Then, for each combination of variables, WALLAP (Strength Factor Method) and PLAXIS were used to perform geotechnical strength assessments using the same variables **and total pile length** per AS5100.3, to determine the "equivalent" factor of safety utilising the Strength Factor method, i.e. FOS_{SF}.

Details of the variables used in each of the parametric models and assessment steps are provided in Table 6 and Table 7.

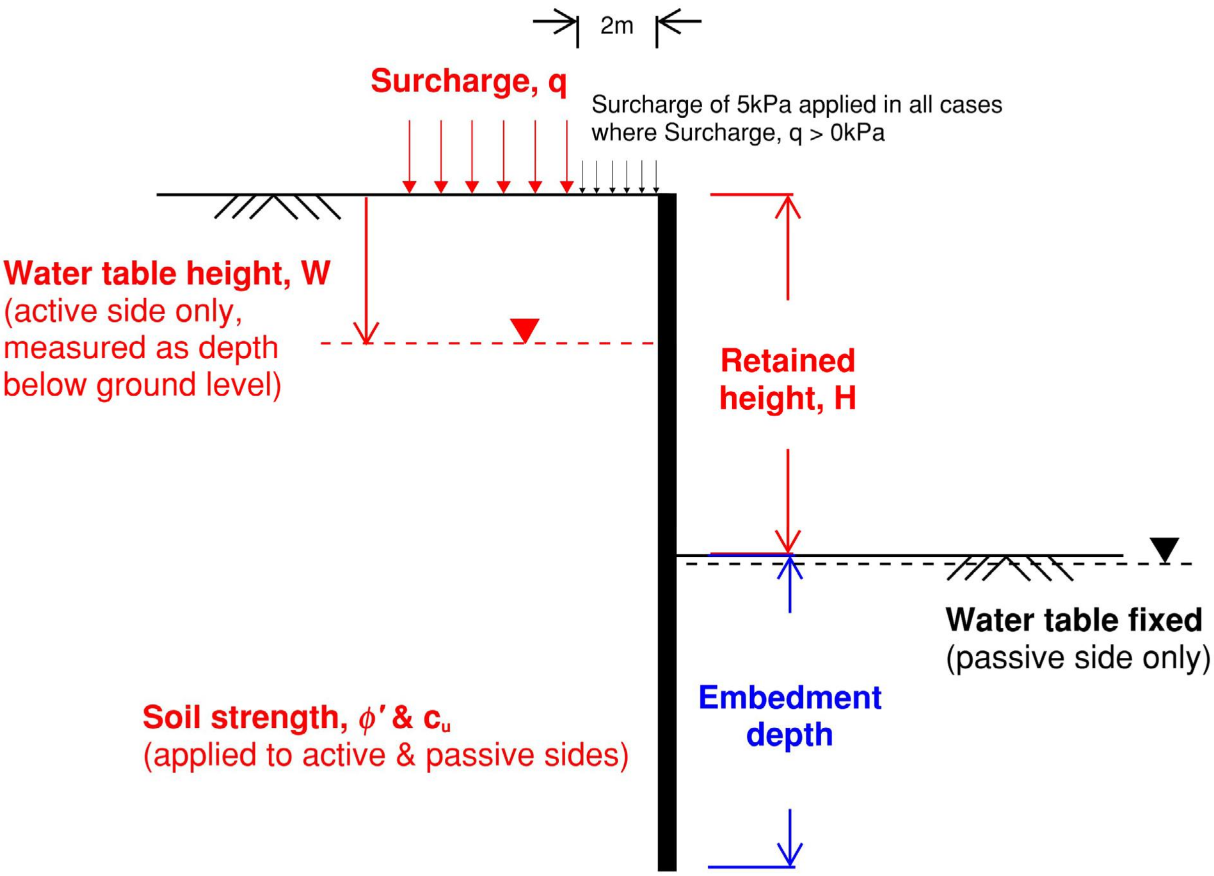


Figure 4: Embedded retaining wall parameters for parametric modelling. Black text = fixed conditions, Red text = parametric variables, Blue text = calculated variable

Table 6: Parametric modelling variables

Tested variable	Range of tested variable	Variables to fix
Soil strength, ϕ'	ϕ' = 24º, 26º, 28º, 30º, 32º, 34º, 36º, 38º	• Soil strength, c' = 5kPa • Retained height = 7.5m • Surcharge = 20kPa • Water level (active) = 1mbgl
Retained height, H	H (m) = 2, 4, 6, 7.5	• Soil strength, c' = 5kPa • Soil strength, ϕ' = 35º • Surcharge = 20kPa • Water level (active) = 1mbgl
Surcharge, q	q (kPa) = 0, 10, 20, 30, 40, 50	• Soil strength, c' = 5kPa • Soil strength, ϕ' = 35º • Retained height = 7.5m • Water level (active) = 1mbgl
Water table (active), W – measured relative to formation level on active side	W (m) = 0, 1.5, 3, 4.5, 6, 7.5	• Soil strength, c' = 5kPa • Soil strength, ϕ' = 35º • Retained height = 7.5m • Surcharge = 20kPa

Table 7: Parametric modelling analysis steps

Step	Assessment Method	Software	Embedment assessment method	Details
1	Method 1	WALLAP *	CP2 (equivalent to AS5100.3)	Select a test variable, e.g. soil strength, ϕ', and apply the first value in the test range. Ensure other variables are fixed
2				Assess necessary embedment depth to achieve ϕ_g per AS5100.3, refer Table 4
3	Method 4	WALLAP	Strength Factor	Replicate the same model from Steps 1 & 2, but select Strength Factor method for analysis
4				Determine FOS achieved (FOS_{SF}) for the same embedment depth of pile calculated using CP2, i.e. Step 2
5	Method 4	PLAXIS2D	Strength Factor (Finite Element)	Replicate the same model from Steps 1 & 2, Run Phi/C reduction (Safety) on the model
				Determine FOS achieved (FOS_{SF}) for the same embedment depth of pile calculated using CP2, i.e. Step 2
6	Repeat Steps 1 – 5 inclusive for next value within the range of test variables, until embedment depths have been calculated for all values within the range			
7	Repeat Steps 1 – 6 inclusive for next test variable, until embedment depths have been calculated for all test variables			

* Note WALLAP software includes options to undertake CP2 analysis methods as well as Strength Factor Methods

7.3 ASSUMPTIONS

The following assumptions were made in the parametric assessments:

- Assessments did not account for the reduction in passive resistance in front of the wall equivalent to 10% of the height above the nominal ground level (minimum of 0.5m), per Clause 8.3.1 in AS5100.3
- Groundwater was modelled assuming hydrostatic groundwater conditions and balanced by interpolating the water pressures linearly based on the groundwater level assigned on each side of the wall separately.
- Assessment was undertaken for geotechnical strength design only. Note that a complete retaining wall design should also include serviceability assessments and structural design, i.e. bending moments and shear forces

7.4 EMBEDDED WALL PROPERTIES

The structural properties of the embedded wall adopted the properties noted below. These properties remained constant in all analyses and were selected to ensure the geotechnical strength solution was not impacted by these properties in any way, i.e. provision of a very stiff wall member. These properties are provided for reference only.

- Construction Material = Contiguous Bored Pile Wall
- Young's Modulus, E = 32,800,000 kPa
- Pile diameter = 1.2m
- Pile spacing = 2.1m centre to centre

7.5 ANALYSIS OUTCOMES

A complete summary of all the parametric modelling outcomes is provided in Table 8. For each parametric model, graphs were plotted to illustrate the calculated outcomes provided in the Appendices. An example is shown in Figure 5.

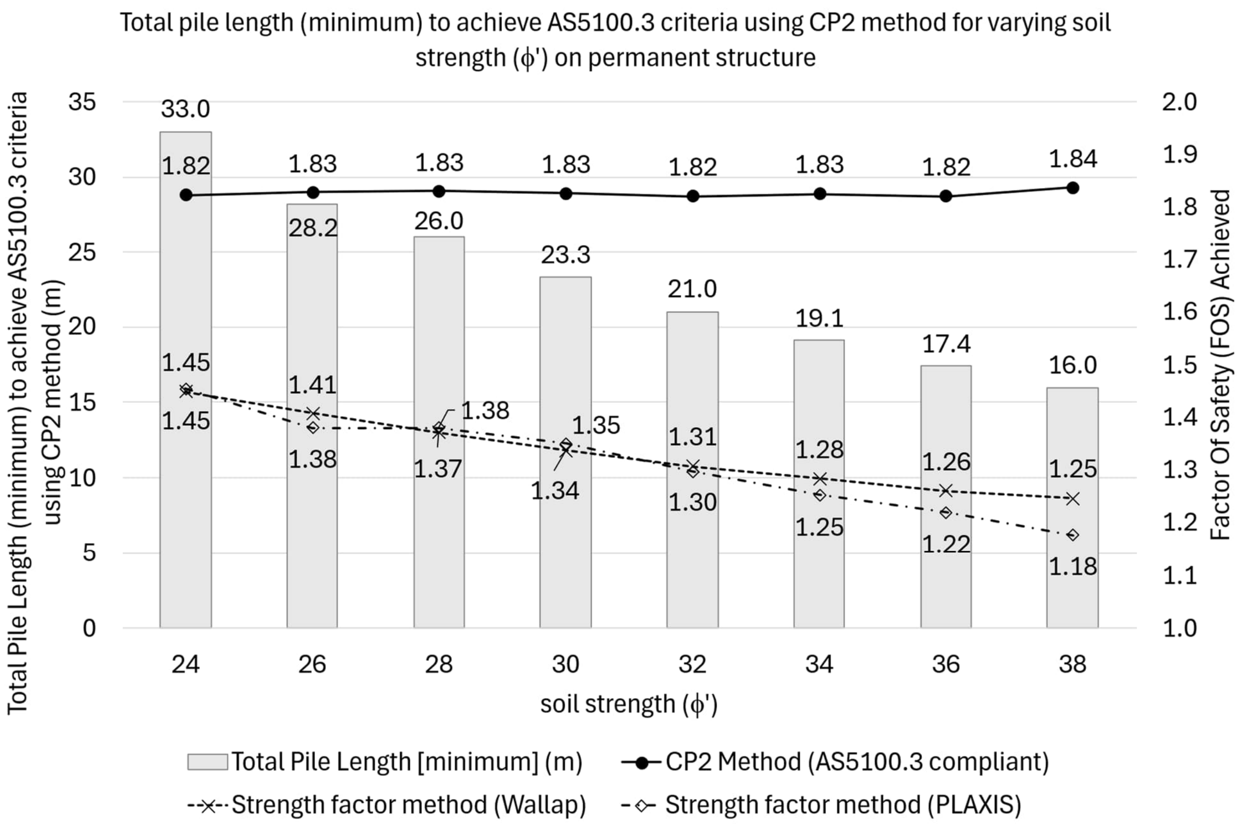


Figure 5: Example of plotted output from parametric model

Table 8: Outcomes of parametric modelling – varying effective soil strength, retained height, surcharge magnitude, water table height

Tested variable	Target FOS_{AS5100}		Equivalent FOS_{SF} using Strength Factor method (based on required pile length to achieve FOS_{AS5100})									
Soil strength, ϕ'		Test variable value (ϕ'), deg	24°	26°	28°	30°	32°	34°	36°	38°	-	Equivalent FOS_{SF}
	1.43	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**27.4**	**24.6**	**22.2**	**20.2**	**18.5**	**16.9**	**15.6**	**14.4**		-
		Strength Factor (WALLAP)	1.23	1.21	1.19	1.18	1.17	1.15	1.14	1.13		1.13 – 1.23
		Strength Factor (PLAXIS Phi/C)	1.25	1.22	1.19	1.17	1.14	1.11	1.07	1.05		1.05 – 1.25
	1.82	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**33**	**28.2**	**26**	**23.3**	**21**	**19.1**	**17.4**	**16**		-
		Strength Factor (WALLAP)	1.45	1.41	1.37	1.34	1.31	1.28	1.26	1.25		1.25 – 1.45
		Strength Factor (PLAXIS Phi/C)	1.45	1.38	1.38	1.35	1.30	1.25	1.22	1.18		1.18 – 1.45
Retained height, H		Test variable value (H), m	2	4	6	7.5	-	-	-	-	-	Equivalent FOS_{SF}
	1.43	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**2.7**	**7.8**	**13.3**	**16.1**	-	-	-	-	-	-
		Strength Factor (WALLAP)	1.10	1.13	1.15	1.15	-	-	-	-	-	1.10 - 1.15
		Strength Factor (PLAXIS Phi/C)	1.12	1.15	1.09	1.07	-	-	-	-	-	1.07 - 1.15
	1.82	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**2.8**	**8.6**	**14.8**	**18.1**	-	-	-	-	-	-
		Strength Factor (WALLAP)	1.14	1.23	1.27	1.27	-	-	-	-	-	1.14 -1.27
		Strength Factor (PLAXIS Phi/C)	1.24	1.26	1.23	1.24	-	-	-	-	-	1.15 - 1.27
Surcharge, q		Test variable value (q), kPa	0	10	20	30	40	50	-	-	-	Equivalent FOS_{SF}
	1.43	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**15.7**	**16**	**16.1**	**16.3**	**16.5**	**16.7**	-	-	-	-
		Strength Factor (WALLAP)	1.15	1.15	1.15	1.15	1.15	1.15	-	-	-	1.15
		Strength Factor (PLAXIS Phi/C)	1.11	1.08	1.07	1.07	1.07	1.06	-	-	-	1.06 - 1.11
	1.82	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**17.6**	**17.9**	**18.2**	**18.4**	**18.6**	**18.8**	-	-	-	-
		Strength Factor (WALLAP)	1.28	1.28	1.28	1.28	1.27	1.27	-	-	-	1.27 - 1.28
		Strength Factor (PLAXIS Phi/C)	1.24	1.23	1.22	1.20	1.19	1.19	-	-	-	1.19 - 1.24

Tested variable	Target FOS_{AS5100}		Equivalent FOS_{SF} using Strength Factor method (based on required pile length to achieve FOS_{AS5100})									
Water table (active), W – measured relative to formation level on active side		Test variable value (W), m	0	1.5	3	4.5	6	7.5	-	-	-	Equivalent FOS_{SF}
	1.43	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**18.5**	**16.9**	**15.5**	**14.3**	**13.3**	**12.6**	-	-	-	-
		Strength Factor (WALLAP)	1.16	1.15	1.13	1.12	1.11	1.10	-	-	-	1.10 - 1.16
		Strength Factor (PLAXIS Phi/C)	1.01	1.03	1.07	1.10	1.09	1.10	-	-	-	1.01 - 1.10
	1.82	**Total pile length to attain FOS_{AS5100} per AS5100.3 (m)**	**20.6**	**18.8**	**17.1**	**15.6**	**14.4**	**13.5**	-	-	-	-
		Strength Factor (WALLAP)	1.29	1.27	1.25	1.22	1.20	1.19	-	-	-	1.19 - 1.29
		Strength Factor (PLAXIS Phi/C)	1.11	1.13	1.16	1.19	1.18	1.16	-	-	-	1.11 - 1.19

8 DISCUSSION OF ANALYSIS OUTCOMES

For parametric models with the same modelled conditions and same embedded wall length,

- CP2 (AS5100.3 compatible) assessment outcomes consistently derived higher FOS compared with the Strength Factor assessments, i.e. FOS_{AS5100} was consistently greater than FOS_{SF}
- The Strength Factor values from both WALLAP and PLAXIS using Strength Factor methods were very similar

The "equivalent FOS" results using Strength Factor methods of analyses (FOS_{SF}) have been summarised in Table 9. Note that this "equivalency" is only relevant for the parameters discussed within this paper.

Table 9: AS5100.3 design criteria and corresponding equivalent FOS using Strength Factor methods (FOS_{SF}) for the same embedment depth. Summarised from Table 8

	AS5100.3 Criteria		Strength Factor method *
Design case	ϕ_g	**FOS_{AS5100}**	**Equivalent FOS (FOS_{SF})**
Temporary structures	0.45 – 0.70	1.43 – 2.22	1.01 - 1.25
Permanent structures	0.45 – 0.55	1.82 – 2.22	1.11 - 1.45

* assessed using Strength Factor method in WALLAP and PLAXIS

9 FUTURE WORK

When this paper was being written, amendment 1 to Part 3 of AS5100.3 – 2017 (Amd 1:2023 Bridge Design Foundation and Soil Supporting Structures) was released in December 2023. Major changes in this amendment affecting stability analysis of retaining walls relate to the range of ϕ_g factor for design of temporary and permanent structures. These are summarised in Table 10.

Table 10: Comparison of design criteria between AS5100.3 – 2017 and AS5100.3 Amd 1:2023

	Comparison of AS5100.3 Criteria	
Design case	**ϕ_g AS5100.3 – 2017**	**ϕ_g AS5100.3 Amd 1:2023**
Temporary structures*	0.45 – 0.70	0.54 – 0.78
Permanent structures	0.45 – 0.55	0.45 – 0.65

*Amd 1:2023 does not provide a ϕ_g for temporary structures but provides guidance that the adopted ϕ_g may be multiplied by a factor of up to 1.2. Provided rages in Table 10 for temporary structures is based on a factor of 1.2 applied to ϕ_g for Permanent Structures.

For simplicity, the analysis undertaken as part of this paper has been based on ϕ_g as per AS5100.3 – 2017 and not the amendment. In authors' view, the conclusions made in this paper remain valid.

10 CONCLUSIONS

The following conclusions can be drawn:

- For geotechnical strength and stability design of embedded retaining walls, AS5100.3 follows a limit state design approach which imposes a reduction factor (geotechnical strength reduction factor, ϕ_g) to the gross passive earth pressure. This approach is analogous to the CP2 method.

- Commonly adopted embedded wall software such as WALLAP (Strength Factor Method) and PLAXIS follows the Strength Factor method which progressively applies a single reduction factor to all values of soil strength in both the active and passive zones until a state of failure or instability is reached
- Due to the incompatibility of the analysis methods, Factors of Safety (FOS) attained using Strength Factor methods (FOS_{SF}) are not equivalent to the FOS (inverse of load / reduction factors) stipulated in AS5100 (FOS_{AS5100})
- For a defined set of parametric variables and assumptions, the authors have ascertained "equivalent FOS" using Strength Factor analyses (Method 4), i.e. FOS_{SF}, to match (as closely as possible) the minimum embedded lengths calculated using CP2 / AS5100.3 (Method 1) which satisfy reduction factors of ϕ_g per AS5100.3, i.e. FOS_{AS5100}. For all parametric models with the with the same modelled conditions and same embedded wall length, CP2 (AS5100.3 compatible) assessment outcomes consistently derived higher FOS compared with the Strength Factor assessments, i.e. FOS_{AS5100} was consistently greater than $FOS_{SF.}$ Note that the "equivalent FOS" values achieved are only applicable for the variables and assumptions covered within this paper, i.e. these values may not be applicable in a universal sense.
- For problems with variables beyond those covered by this paper, it is the authors' opinion that AS5100.3 (CP2) assessments should be undertaken in the first instance to ensure attainment of the prescribed project criteria before embarking on Strength Factor methods using more commonly used software.
- It is essential that designers recognise the different methods available for embedded wall stability assessment and understand that FOS is defined differently across these methods.
- This paper addresses the criteria from AS5100.3 – 2017 and AS5100.3 Supplement 1 – 2008 only.
- This paper addresses geotechnical strength and stability analyses only. Serviceability and Structural design are not covered herein but should always be considered for all retaining wall designs

CRediT authorship contribution statement

Idy Li: Writing - original draft. **Jawad Zeerak:** Writing - original draft. **Jackson Ho:** Writing - original draft.

11 REFERENCES

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CIRIA C760 (2017) Guidance on embedded retaining wall design
Clayton C.R.I, Woods R.I, Bond A.J, Milititsky J (2013) Earth Pressure and Earth-Retaining Structures, Third Edition, CRC Press
PLAXIS Connect Edition V22.01 (2022) Finite element code for soil and rock analyses, Version 22, Bentley

12 APPENDICES

Graphs of all parameter models

- Tested variable, Soil strength, ϕ' (range = 24° to 38°)
- Retained height, H (range 2m to 7.5m)
- Surcharge pressure, q (range 0kPa to 50kPa)
- Water table height, W (range 0m to full retained height)

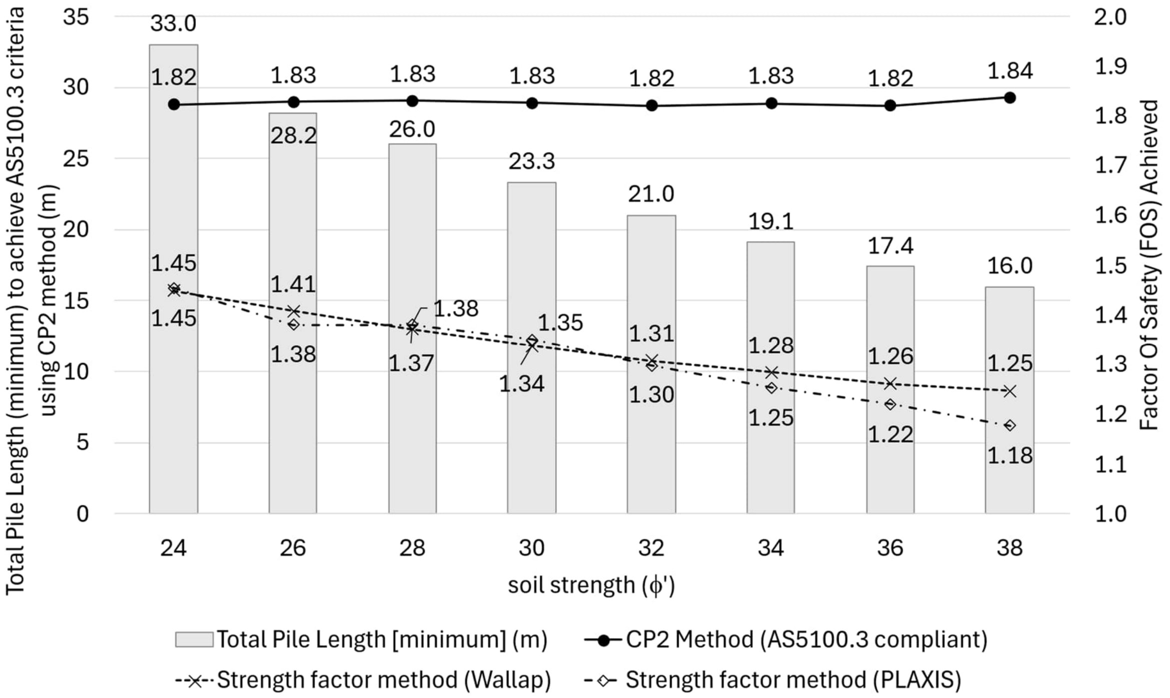

Figure 6: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying soil strength (ϕ) on permanent structure

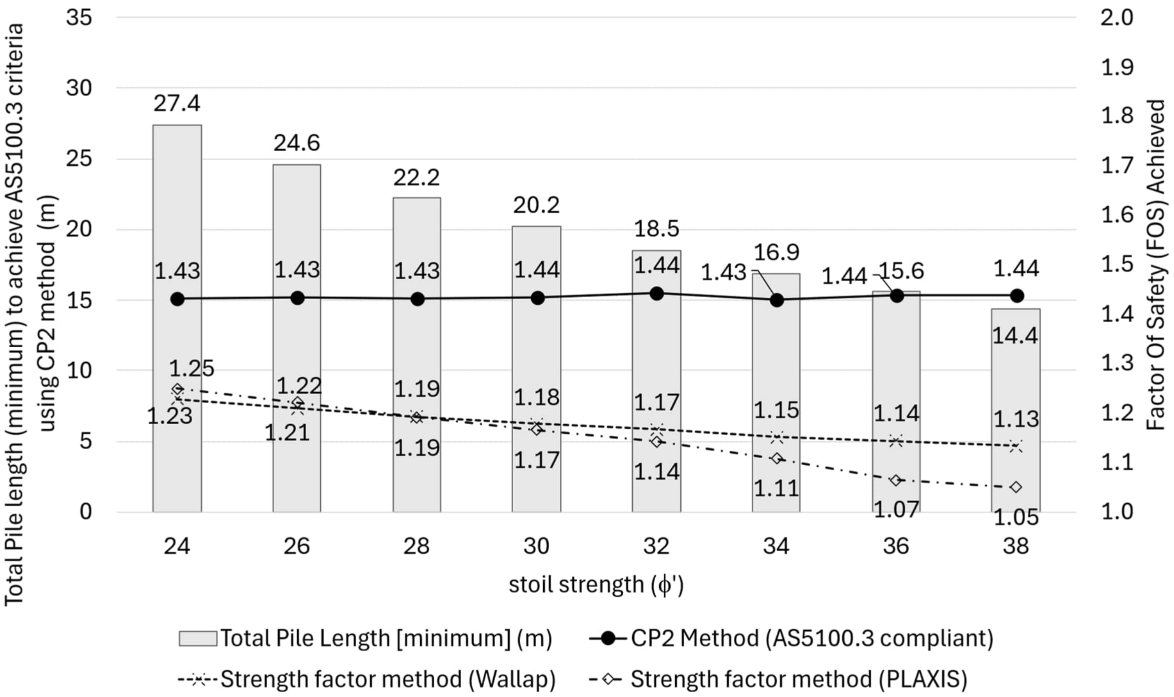

Figure 7: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying soil strength (ϕ) on temporary structure

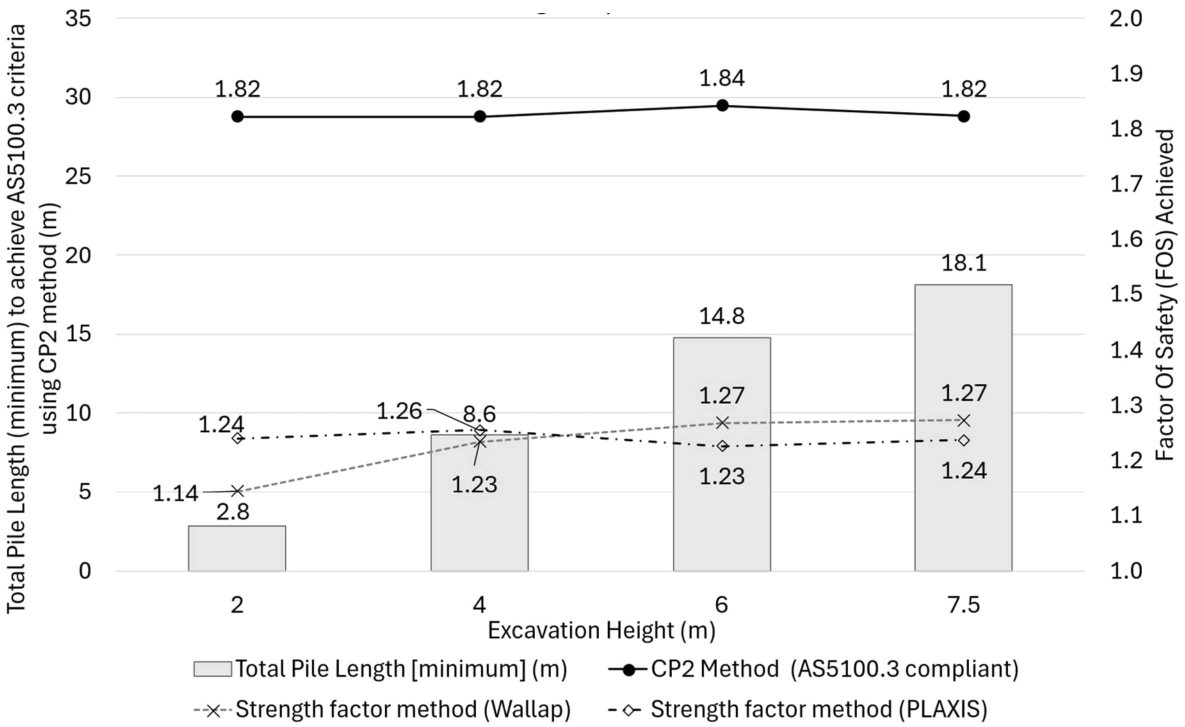

Figure 8: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying excavation height on permanent structure

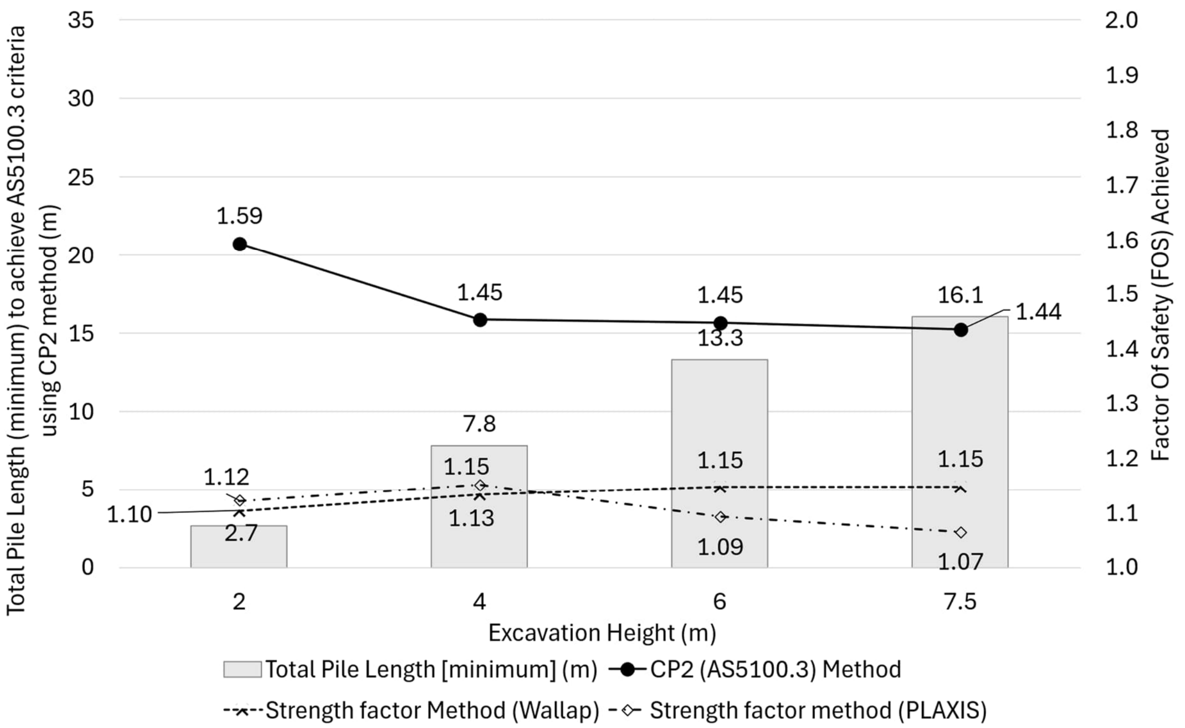

Figure 9: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying excavation height on temporary structure

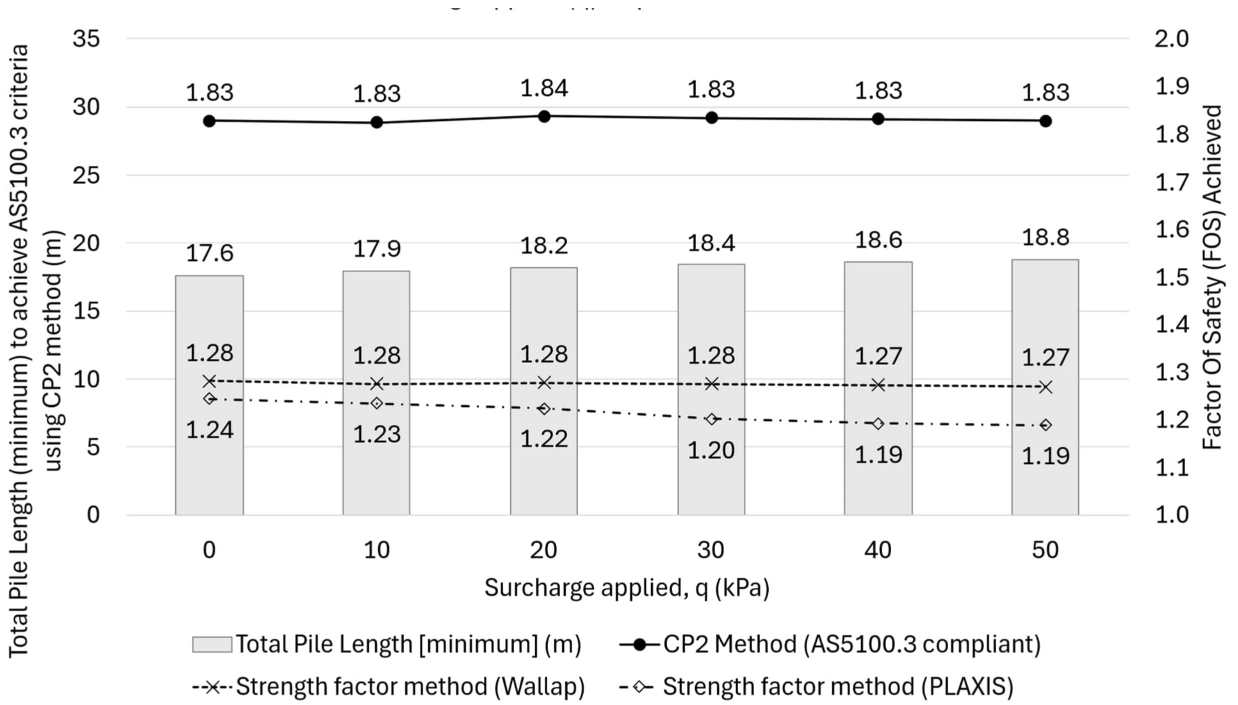

Figure 10: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying surcharge applied (q) on permanent structure

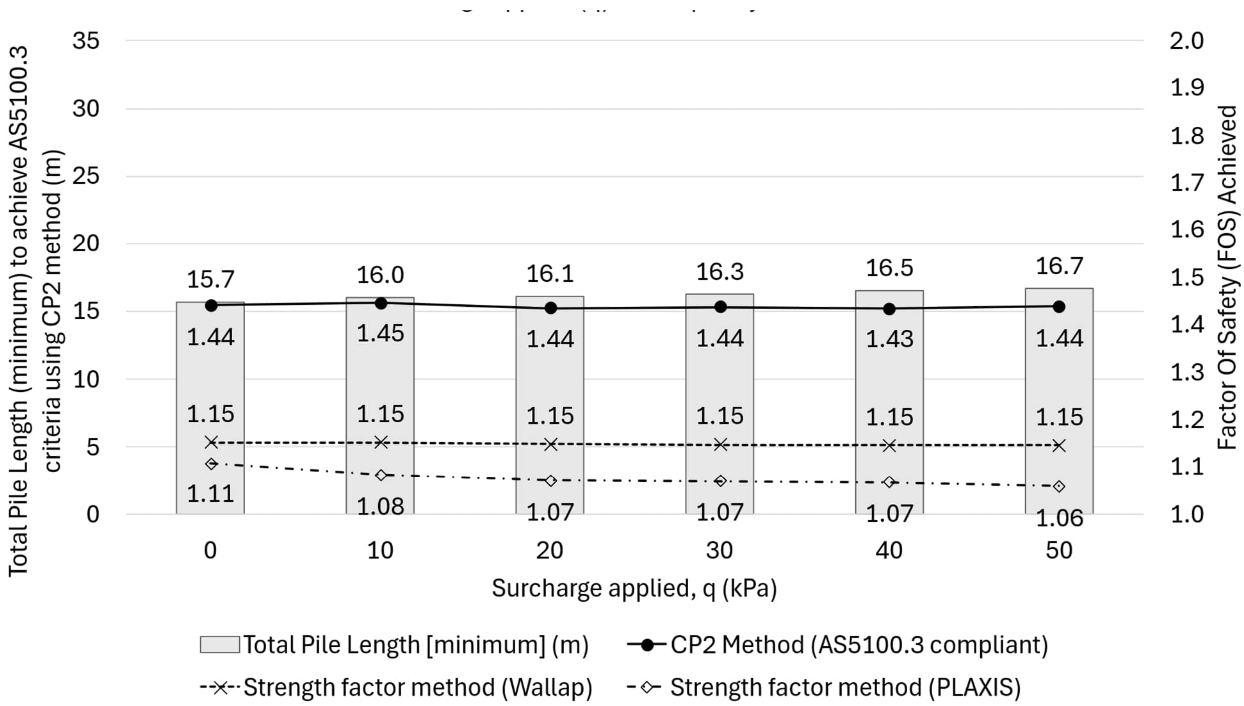

Figure 11: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying surcharge applied (q) on temporary structure

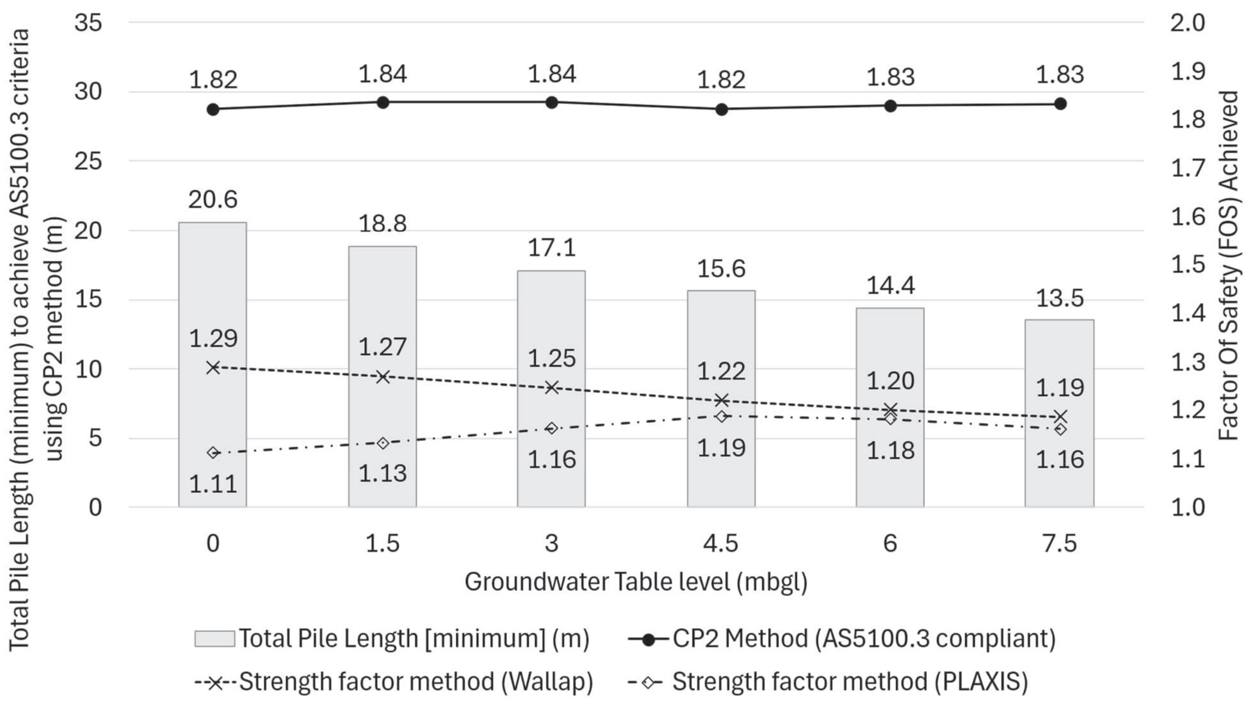

Figure 12: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying groundwater table level on permanent structure

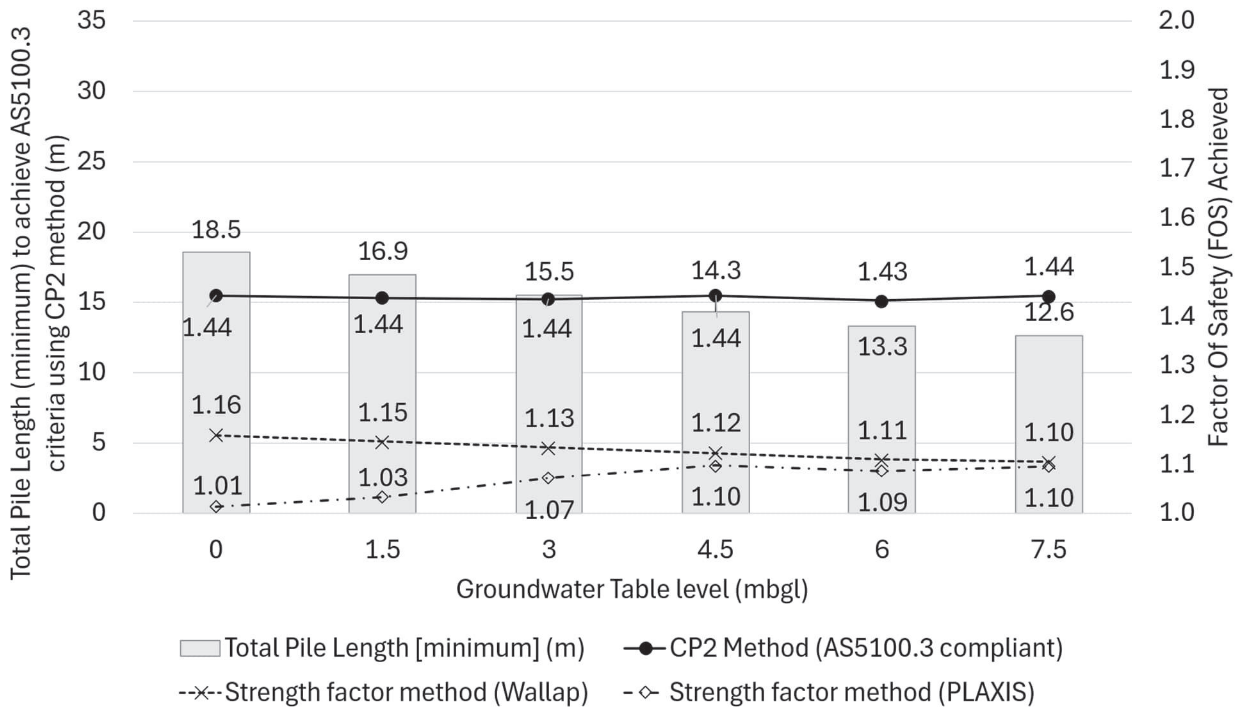

Figure 13: Total pile length (minimum) to achieve AS5100.3 criteria using CP2 method for varying groundwater table level on temporary structure

NOTES ON HIGH HORIZONTAL STRESS GENERATION AND POST PROCESSING FOR DEEP EXCAVATION SHORING WALL USING PROGRAM PLAXIS 2D

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https://doi.org/10.56295/AGJ6125

ABSTRACT

With recent advancement of computer technology and software engineering, numerical modelling is becoming an essential tool to carry out a soil structure interaction analysis of a shoring system for deep excavation. This technical note has been prepared to provide specific approaches for a shoring system design of a deep excavation in Sydney using commercially available software Plaxis. The first part of this note covers a method of generating high in-situ horizontal stresses in Plaxis model, which are usually presented in Sydney Sandstone and Shale formation. The second part presents a useful tool for post-processing of analysis results from Plaxis 2D for soil structure interaction analysis of shoring walls for deep excavation.

1 INTRODUCTION

Numerical modelling using commercially available software Plaxis has been widely used in geotechnical engineer, including the Sydney Sandstone and Shale where high locked-in in-situ horizontal stresses are often present. The default option in Plaxis 2D for high in-situ stress within rock is to apply a homogenous initial stress state in the model with the filed stress option. Where there is a case with horizontal stresses increase with depth it would require the user to physically define the input for each element with depth. This is often found inconvenient, hence there is a need to develop a methodology for auto-generation of the high in-situ horizontal stresses for Plaxis modelling.

During the detailed design of M4 East project (WestConnex Stage 1B) in Sydney and construction process in 2015, the authors developed an approach to generating the in-situ high horizontal stresses in rock using an applied volume strain when program Plaxis 2D and Plaxis 3D are used. Since successful application of this approach, it has subsequently been widely adopted by many practicing geotechnical engineers for Plaxis modelling over the past 10 years, notably Sydney Metro Northwest and Sydney Metro City, Sydney West Metro, Western Sydney Airport Metro, New M5 Motorway, M4-M5 Link and Rozelle Interchange in Sydney and Melbourne Metro in Melbourne. The first part of this note will set out the details of how this methodology can be used for ease of reference.

It is very important to have efficient communications between geotechnical and structural engineers for shoring wall design of a deep excavation. The role of geotechnical engineers is often to carry our soil and structure interaction analysis and then pass the analysis results to structural engineers to carry out the structural design such as reinforced concrete structure or steel members such as steel props and waler beams.

Preparation of the analysis results for the shoring walls sounds simple but could be a very time-consuming process, especially nowadays the construction sequences or stages are becoming increasingly complex under urban development environment. When many construction stages (for example, more than 30 analysis stages) are modelled in the numerical analysis, the load path transfer will become very complex and therefore the critical stages where the design can be focused on will become less obvious. This situation always present technical challenges to the geotechnical engineers who are responsible for providing analysis results such as bending moment, shear force, axial force and deflection within the retaining wall structures which are modelled by the plate elements in Plaxis.

The second part of this note introduces the geotechnical automation works we are currently undertaking within Arcadis to streamline the interaction process between geotechnical and structural engineers for cut and cover structure design. This process is currently being used for multiple infrastructure the projects across Australia and has been proved useful. This note is to share the approach and written scripts with the geotechnical colleagues who may wish to use for their future projects.

2 MODELLING METHODOLOGY AND PROCEDURES

The high in-situ stresses have been well recognised from the construction of tunnels and caverns in Sydney Region, with some of the published works by Enever (1990), Pells (1990) and McQueen (2004). The high in-situ horizontal stresses have often caused the shear movements along the bedding planes, joints or sheared zones, which, in turn, result in damages to the installed support system such as rock bolts or shotcrete in tunnels and deep excavations. Therefore, it is essential to simulate this high horizontal stress properly when a numerical modelling is undertaken.

In the Sydney region, the rock classification system (Class I to Class V) proposed by Pells et al. (1998) is commonly adopted for Hawkesbury Sandstone and Ashfield Shale, based on rock strength, defect spacing, and the presence of allowable seams.

Typical in-situ horizontal stresses in Sydney Sandstone Class I/II are commonly recommended to be as follows:

$$\boldsymbol{\sigma}_h = (1.5\sim2.5) + 2\boldsymbol{\sigma}_v \text{ (MPa)} \quad (1)$$

where $\boldsymbol{\sigma}_h$ is the horizontal stress while $\boldsymbol{\sigma}_v$ is the vertical stress. Equation (1) shows that the recommended in-situ horizontal stresses within Class I/II rocks consists of two parts: (1) constant component of 1.5 ~2.5 MPa within the rock and (2) linearly increased component of with depth, $2\sigma_v$, which is related to the vertical stresses within the rock mass.

It should be pointed out that gravity loading method in PLAXIS 2D cannot generate in-situ stress with k_0 greater than 1 because the value of k_0 is directly related to the Poisson's ratio, ν, by the following equation:

$$k_0 = \nu/(1- \nu) \quad (2)$$

An alternative method based on volumetric strain is therefore proposed to generate the exact in-situ horizontal stress distribution within rock mass as recommended in Equation (1). This method is based on the following relationship between the stress and strain obeying Hooke's Laws for the rock mass and can be used to generate the additional horizontal stress, σ_{xx}, which is the first component, σ_h, of Equation (1). Modification of Poisson's ratio will be required as discussed in Step 2 below.

$$\boldsymbol{\sigma}_{xx} = E\varepsilon_{xx} + \nu(\boldsymbol{\sigma}_{yy} + \boldsymbol{\sigma}_{zz}) \quad (3)$$

where E is Young's modulus, $\boldsymbol{\sigma}_{yy}$ and $\boldsymbol{\sigma}_{zz}$ are horizontal stress in y and z directions, respectively, ε_{xx} is the horizontal strain in x direction.

The detailed procedures to generate high in-situ horizontal stresses within rock mass are described as follows:

Step 1: Generating the initial in-situ horizontal stresses within rocks with adopting k_0-procedure and k_0 value of 2 (the second component of Equation-1).

Step 2: Immediately after Step 1, plastic analysis stage is then carried out with assigning a specified horizontal volume strain ε_{xx} to Sandstone/Shale Class I/II rock layers in the model. Additional horizontal stresses (the first component of Eqiation-1) will be then generated and added onto the existing in-situ horizontal stresses calculated from the first step within the soils and rocks. The input value for the horizontal volume strain value will need to be calculated based on the following equation:

$$\varepsilon_{xx} =1/\{E(\boldsymbol{\sigma}_{xx} – \nu(\boldsymbol{\sigma}_{yy} + \boldsymbol{\sigma}_{zz}))\} \quad (4)$$

A very small value should be adopted for the Poisson's ratio for the rock (Class I/II) materials in order to reduce the effects from other stress components. For example, additional horizontal stress of 1.5 MPa corresponds to a horizontal strain of 0.075% for the rock with Young's modulus of 2000 MPa. It should be pointed out that the Poisson's ratio needs to be reset to the typical value of 0.2 for the subsequent analyses after the generation of in-situ stresses.

The modelling workflow outlined in this paper was originally developed in 2015 during detailed design development for the M4 East project, WestConnex Stage 1B in Sydney, Australia. It is worth noting that a similar approach was presented in a technical post for Plaxis Modelling by Bentley Systems (2024). This paper provides the theoretical background and derivation of the equations based on Hooke's law, together with a more comprehensive description of the detailed procedures and worked examples. As such, this paper provides a useful technical reference in geotechnical numerical modelling.

3 WORKED EXAMPLE

A worked example problem has been used to demonstrate the application of the proposed method for deep excavation into Sandstone/Shale Class I/II rock mass. The strength and deformation parameters used for the rock mass are summarised in the table below:

Table 1: Input parameters for rock mass

Unit weight (kN/m^3)	Effective cohesion (kPa)	Effective angle of friction (deg)*	Young's modulus (MPa)	Poison's ratio	Tensile strength (kPa)
24	500	36	2000	0.2	50

The geometry of the deep excavation into Class I/II rocks with high in-situ horizontal stressed is shown in Figure 1. The total excavation depth is 8 m in this analysis case and a 20 kPa surcharge is applied at the ground surface. Figure 2 shows the distribution of in-situ horizontal stresses within the rock from the first step (Step 1) of the in-situ stress generation procedure. The in-situ horizontal stresses are calculated by multiplying the vertical stresses with the specified k_0 value (k_0=2 for this case). Note that the in-situ horizontal stress is 0 at the ground surface from this step. Figure 3 shows the in-situ horizontal stress distribution from the second step (Step 2) after the volume strain in the horizontal direction is applied. It can be seen that the in-situ horizontal stress is 1500 kPa at the ground surface (this cannot be obtained by using k_0-procedure alone) and increases linearly with the depth.

Figure 4 shows the comparison results between the recommended linear relationship and analysis results from PLAXIS 2D based on the proposed volume strain approach. It can be seen that in-situ horizontal stresses commonly encountered in Sydney can be correctly modelled in PLAXIS 2D through the proposed volumetric strain approach.

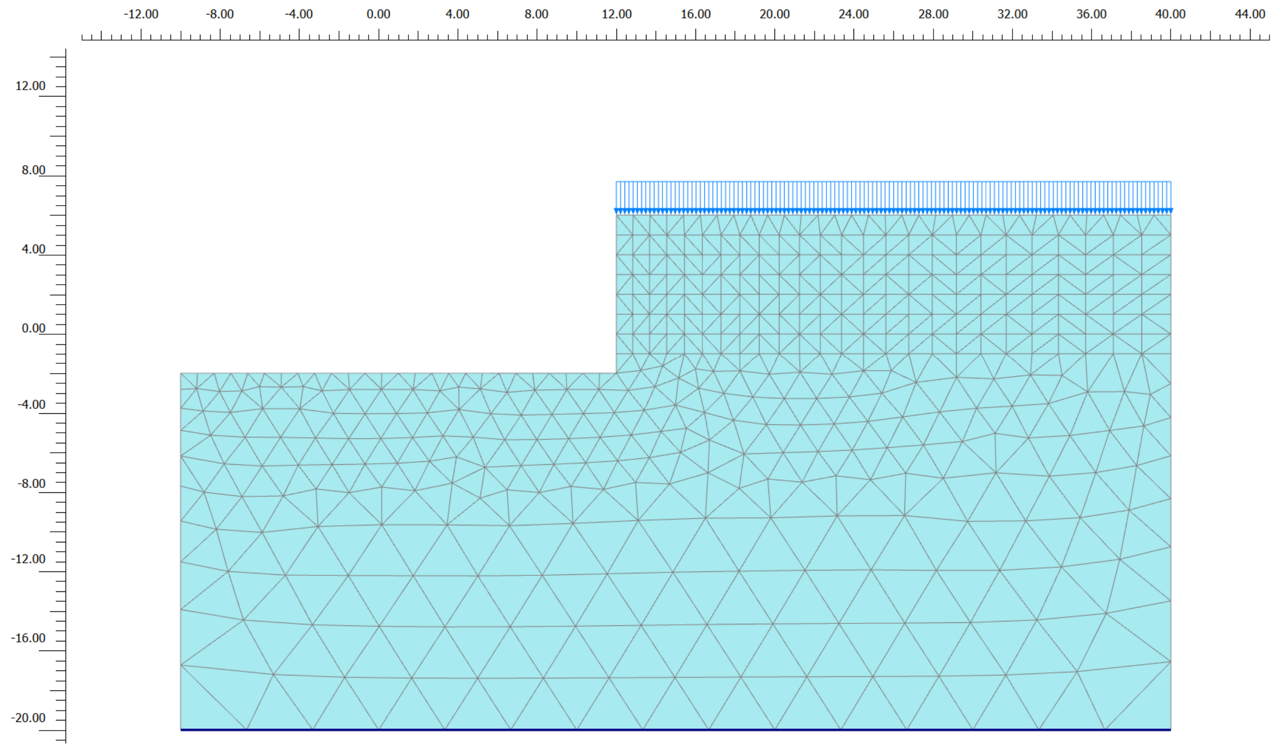


Figure 1: Mesh generated by Plaxis 2D

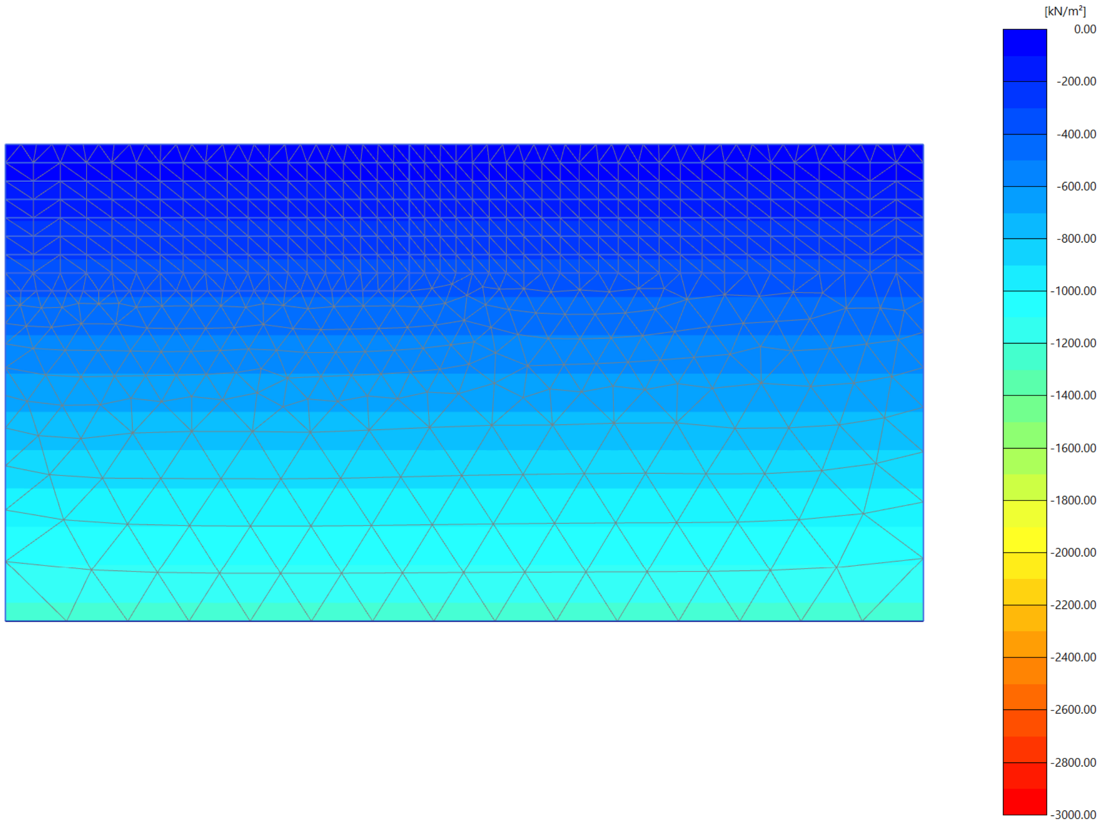


Figure 2: In-situ horizontal stress contours generated in Step 1 using k_0 procedure

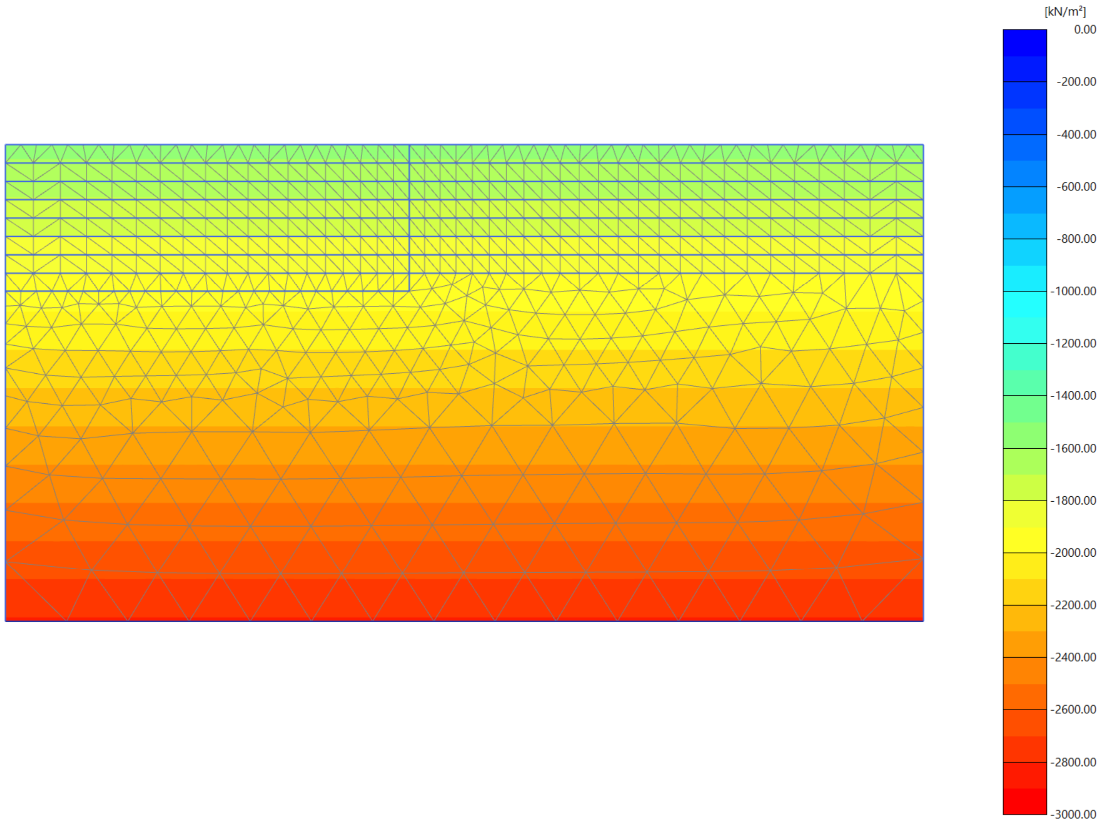


Figure 3: Horizontal stress contours generated by Plaxis 2D

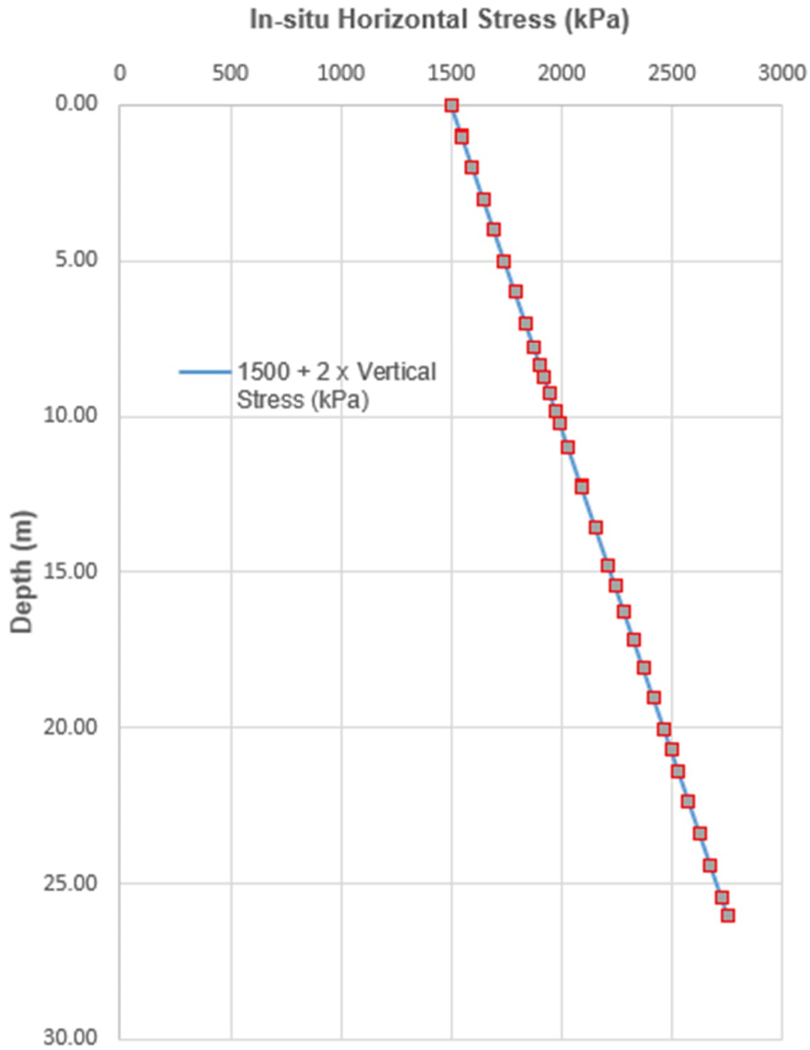


Figure 4: Comparisons of in-situ horizontal stresses from the recommended linear relationship and analysis results from PLAXIS 2D based on the proposed volume strain approach

4 POSTPROCESSING OF PLAXIS 2D ANALYSIS RESULTS

Post processing of Plaxis 2D analysis results for the shoring wall for deep excavations could be a time-consuming task when multiple construction stages are required for simulation. As part of geotechnical automation innovation process, a python-based tool named PlaxisPL2D has been developed to streamline the post-processing of analysis results from Plaxis 2D.

By using PlaxisPlate2D, engineers and geotechnical professionals can quickly export and present the analysis results from Plaxis 2D in a desirable format by reducing significant amount of the post analysis processing time. Furthermore, PlaxisPlate2D will eliminate potential human errors of repeated data processing. PlaxisPlate2D is written in Python, which is a powerful and widely used programming language, and it is specifically designed for deep excavation analysis where plate elements are used to model the shoring walls. PlaxisPL2D can be made available free upon request via an email to the authors.

PlaxisPlate2D can be directly run from the Plaxis output program as shown in Figure 5. Some of the key features and benefits from PlaxisPlate2D are summarised as follows:

- PlaxisPlate2D extracts the Plaxis 2D analysis results for all the plate elements and node-to-node anchors/structs within the model and present the results in professionally designed charts and tables which can be directly used in the design report or for the presentation.
- The extracted results will be automatically stored in a single Microsoft Excel Spreadsheet file.

- It generates charts for the results of displacement, bending moment, shear force and axial force of plate elements with depth/level for all the analysis stages.
- It can be used for the analysis and design for various deep excavation problems where the retaining walls are modelled with plate elements.
- It automatically generates the summary table which presents the results of the calculated actions in wall and associated ground anchors for every analysis stage.

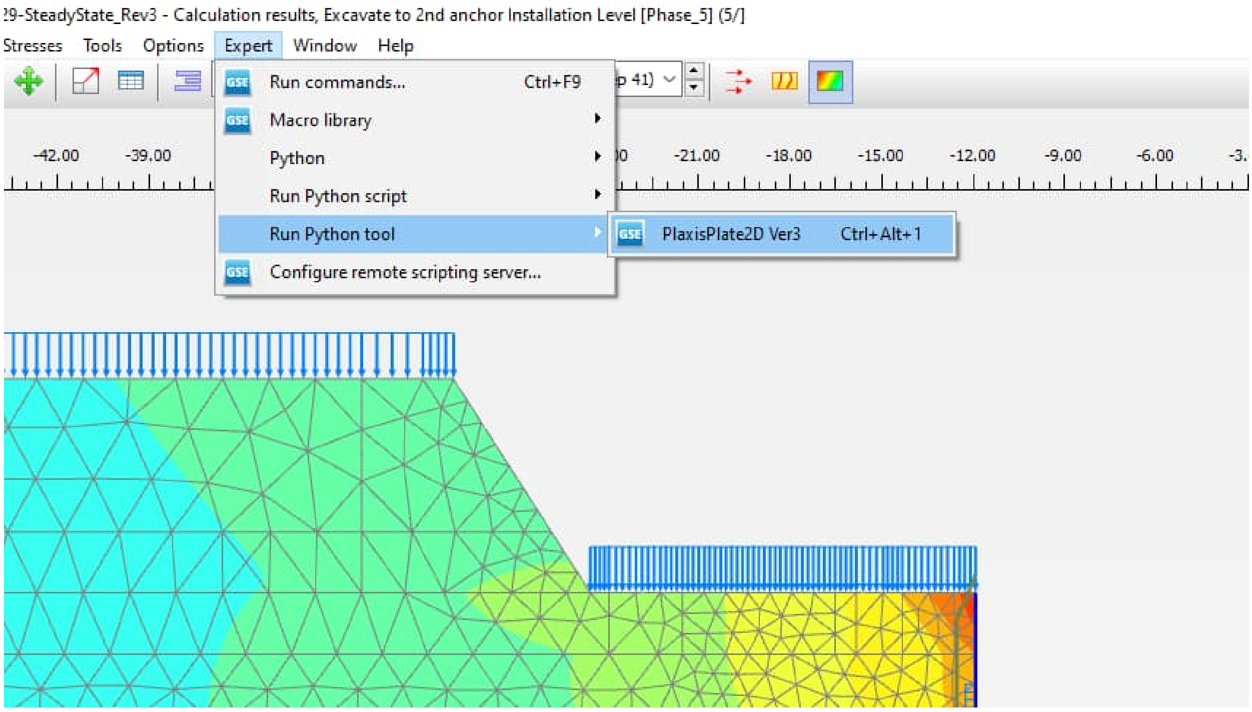


Figure 5: PlaxisPlate2D tool within Plaxis 2D output program

The user interface for this tool is shown in Figure 6. The required inputs to extract the plate results are summarised as below:

- X coordinate of the vertical plate member in Plaxis 2D. Note that this tool only can extract the results for one continuous vertical plate member only. Currently, it does not support the horizontal and inclined plate members,
- Min Y coordinate of the vertical plate member in Plaxis 2D. This represents the lowest point for the vertical plate member. This does not need to be within the plate member and can be below the vertical plate member. The tool will automatically check the valid plate elements which are above this point.
- Max Y coordinate of the vertical plate member in Plaxis 2D. This represents the highest point for the vertical plate member. This does not need to be within the plate member and can be above the vertical plate member. The tool will automatically check the valid plate elements which are below this point.
- Port ID. This is the port ID used when the Plaxis remote scripting server is started.
- Password: This is the password used when the Plaxis remote scripting server is enabled.
- Plate name: This is the plate name which can be defined by the users before the result extraction. This name will be included into the Excel spreadsheet file.
- Select folder: This is to select the folder where the extracted data file will be saved to. Clicking the button of “Browse” will help to select the folder as the user wishes. The user can directly enter the full folder path.
- Spreadsheet name: This is the name of the Excel spreadsheet file created by PlaxisPlate2D.

The result extraction progress will be displayed under the “Progress” section when the “Start Importing” button is clicked.

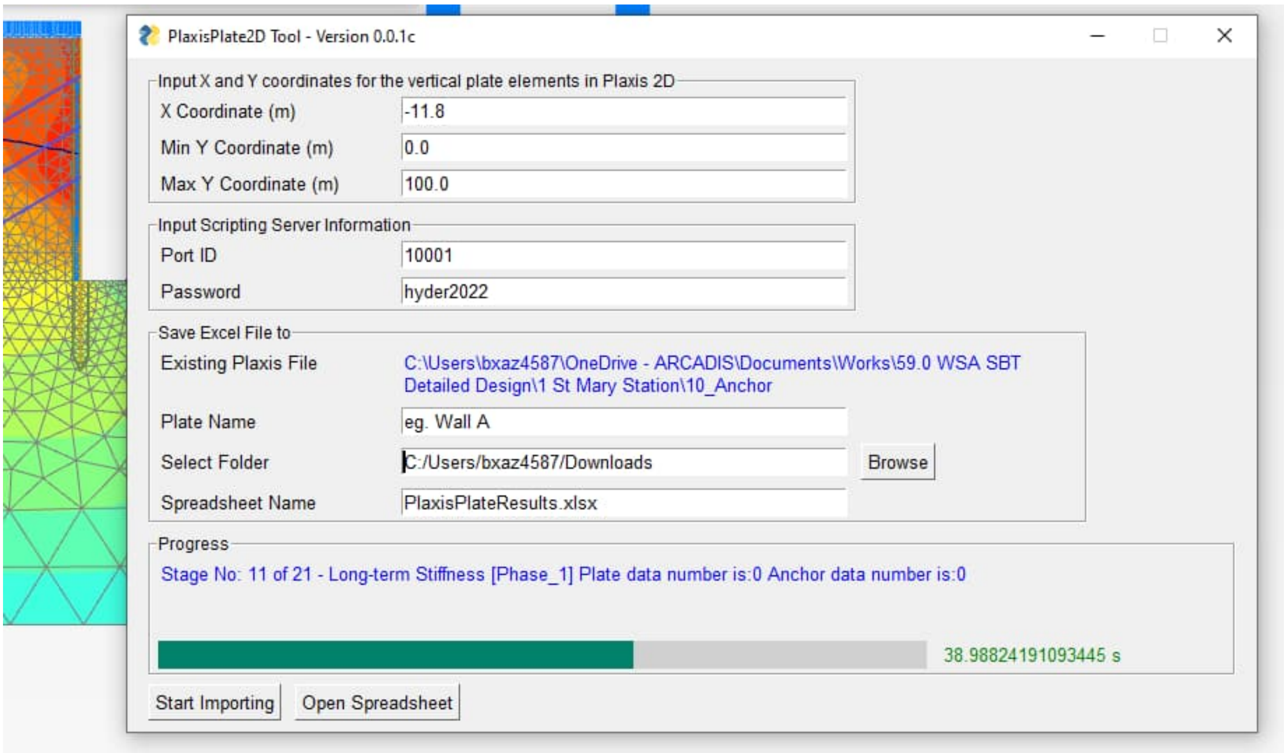


Figure 6: PlaxisPlate2D user interface

A typical spreadsheet output is shown in Figure 7. Multiple sheets are included in the spreadsheet file. The first three sheets are "About", "Summary" and "Plots", followed by individual sheets which includes the detailed results for each analysis stage. "About" sheet shows the general information for the selected plates and the output file.

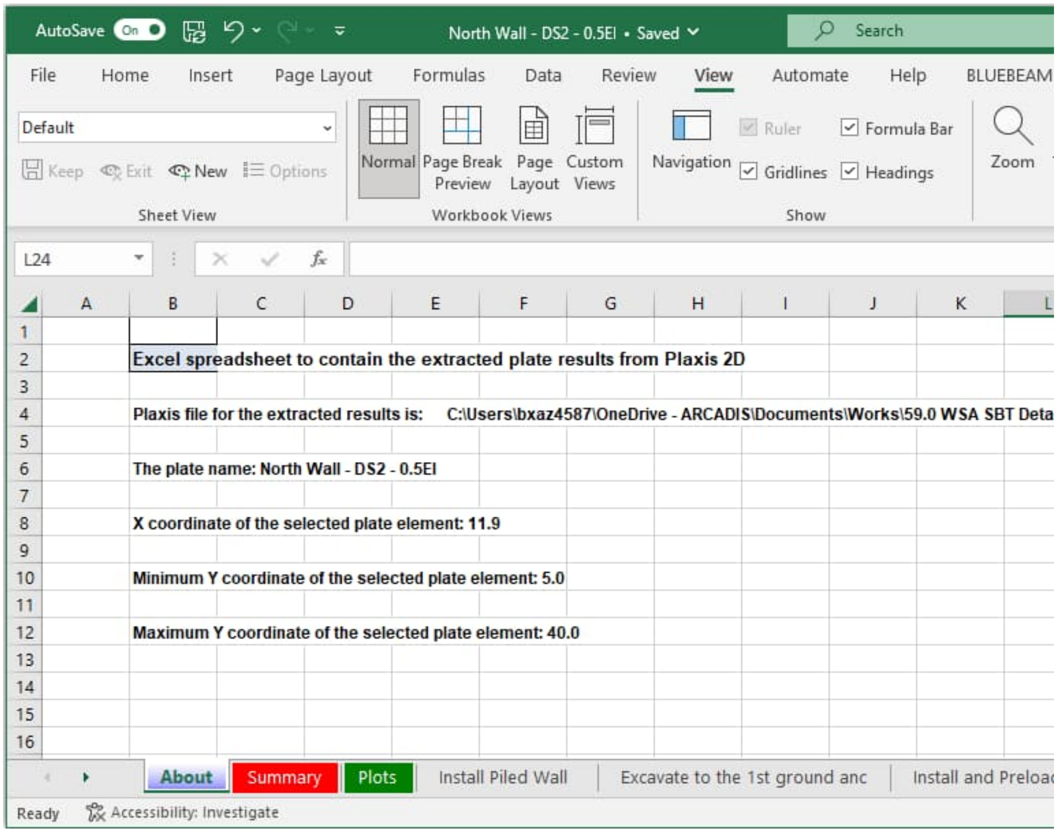
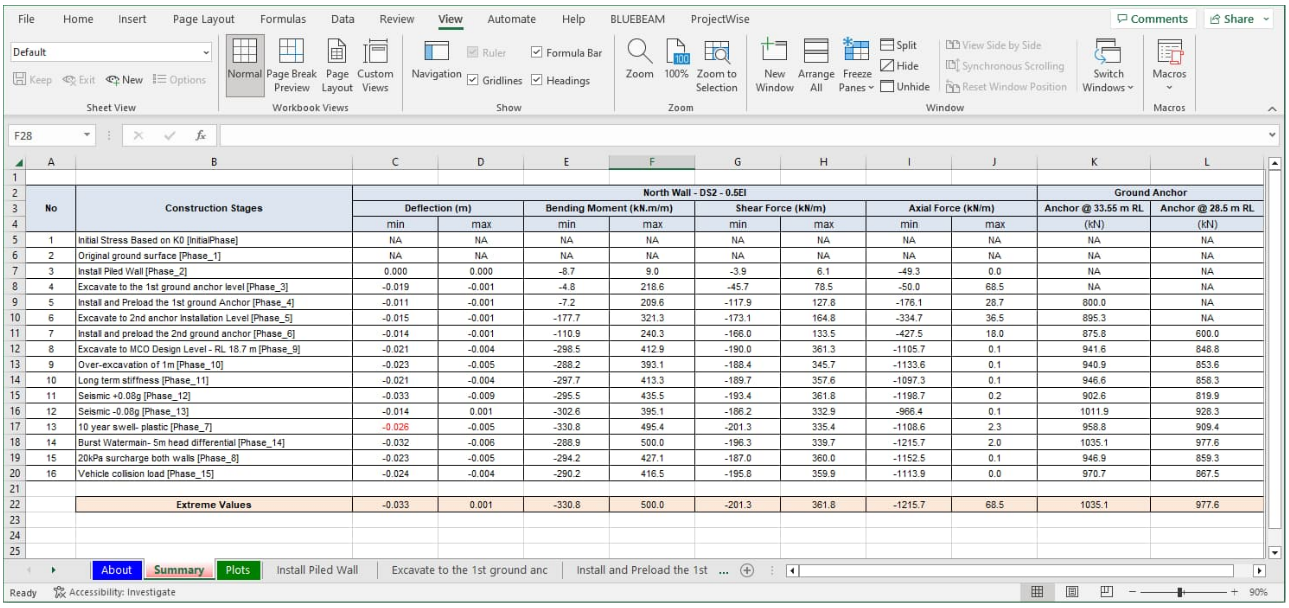


Figure 7: PlaxisPlate2D output file – "About" sheet

Figure 8 shows the "Summary" sheet which presents the summarised results of the calculated actions in wall and associated ground anchors (which shares the same X coordinate with the plate) for every analysis stage for this example case. The minimum and maximum values are also provided.



No	Construction Stages	North Wall - DS2 - 0.5EI								Ground Anchor	
		Deflection (m)		Bending Moment (kN.m/m)		Shear Force (kN/m)		Axial Force (kN/m)		Anchor @ 33.55 m RL	Anchor @ 28.5 m RL
		min	max	min	max	min	max	min	max	(kN)	(kN)
1	Initial Stress Based on K0 [InitialPhase]	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
2	Original ground surface [Phase_1]	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
3	Install Piled Wall [Phase_2]	0.000	0.000	-8.7	9.0	-3.9	6.1	-49.3	0.0	NA	NA
4	Excavate to the 1st ground anchor level [Phase_3]	-0.019	-0.001	-4.8	218.6	-45.7	78.5	-50.0	68.5	NA	NA
5	Install and Preload the 1st ground Anchor [Phase_4]	-0.011	-0.001	-7.2	209.6	-117.9	127.8	-176.1	28.7	800.0	NA
6	Excavate to 2nd anchor Installation Level [Phase_5]	-0.015	-0.001	-177.7	321.3	-173.1	164.8	-334.7	36.5	895.3	NA
7	Install and preload the 2nd ground anchor [Phase_6]	-0.014	-0.001	-110.9	240.3	-166.0	133.5	-427.5	18.0	875.8	600.0
8	Excavate to MCO Design Level - RL 18.7 m [Phase_9]	-0.021	-0.004	-298.5	412.9	-190.0	361.3	-1105.7	0.1	941.6	848.8
9	Over-excavation of 1m [Phase_10]	-0.023	-0.005	-288.2	393.1	-188.4	345.7	-1133.6	0.1	940.9	853.6
10	Long term stiffness [Phase_11]	-0.021	-0.004	-297.7	413.3	-189.7	357.6	-1097.3	0.1	946.6	858.3
11	Seismic +0.08g [Phase_12]	-0.033	-0.009	-295.5	435.5	-193.4	361.8	-1198.7	0.2	902.6	819.9
12	Seismic -0.08g [Phase_13]	-0.014	0.001	-302.6	395.1	-186.2	332.9	-966.4	0.1	1011.9	928.3
13	10 year swell- plastic [Phase_7]	-0.026	-0.005	-330.8	495.4	-201.3	335.4	-1108.6	2.3	958.8	909.4
14	Burst Watermain- 5m head differential [Phase_14]	-0.032	-0.006	-288.9	500.0	-196.3	339.7	-1215.7	2.0	1035.1	977.6
15	20kPa surcharge both walls [Phase_8]	-0.023	-0.005	-294.2	427.1	-187.0	360.0	-1152.5	0.1	946.9	859.3
16	Vehicle collision load [Phase_15]	-0.024	-0.004	-290.2	416.5	-195.8	359.9	-1113.9	0.0	970.7	867.5
	Extreme Values	-0.033	0.001	-330.8	500.0	-201.3	361.8	-1215.7	68.5	1035.1	977.6

Figure 8: PlaxisPlate2D output file – "Summary" sheet

Figure 9 shows the plots for deflection, bending moment of the plate member against the reduced level. The results for all the analysis stages modelled are included.

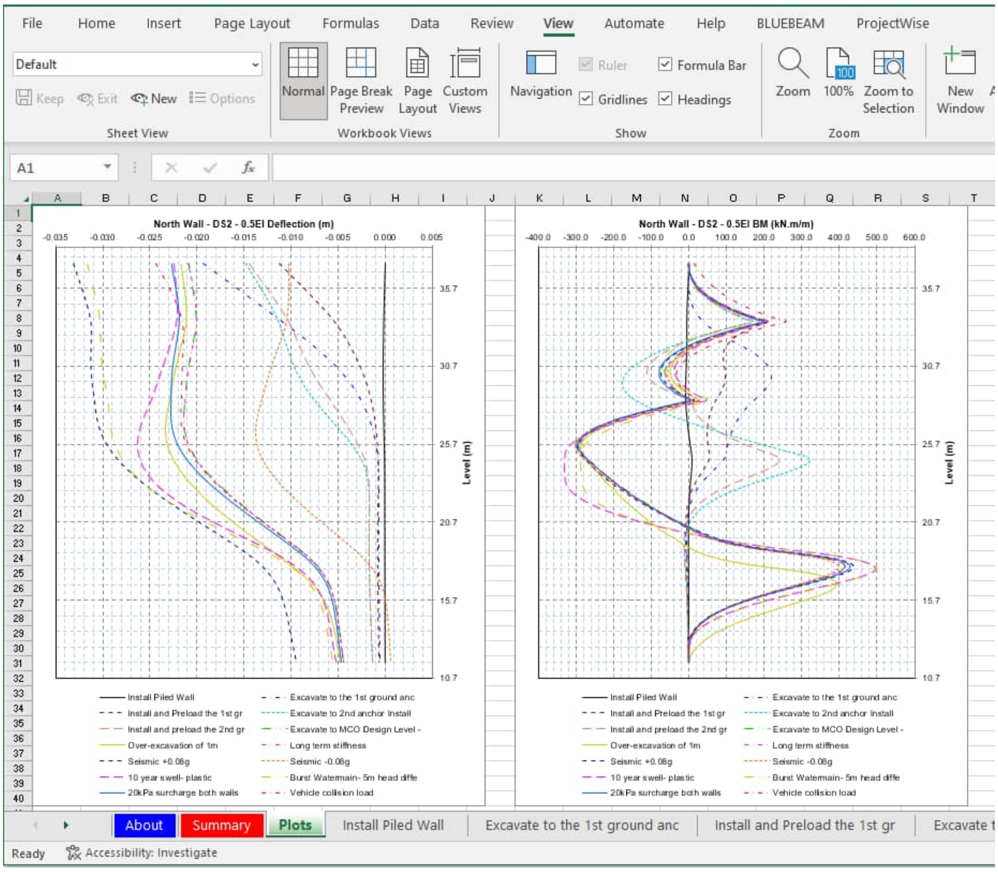
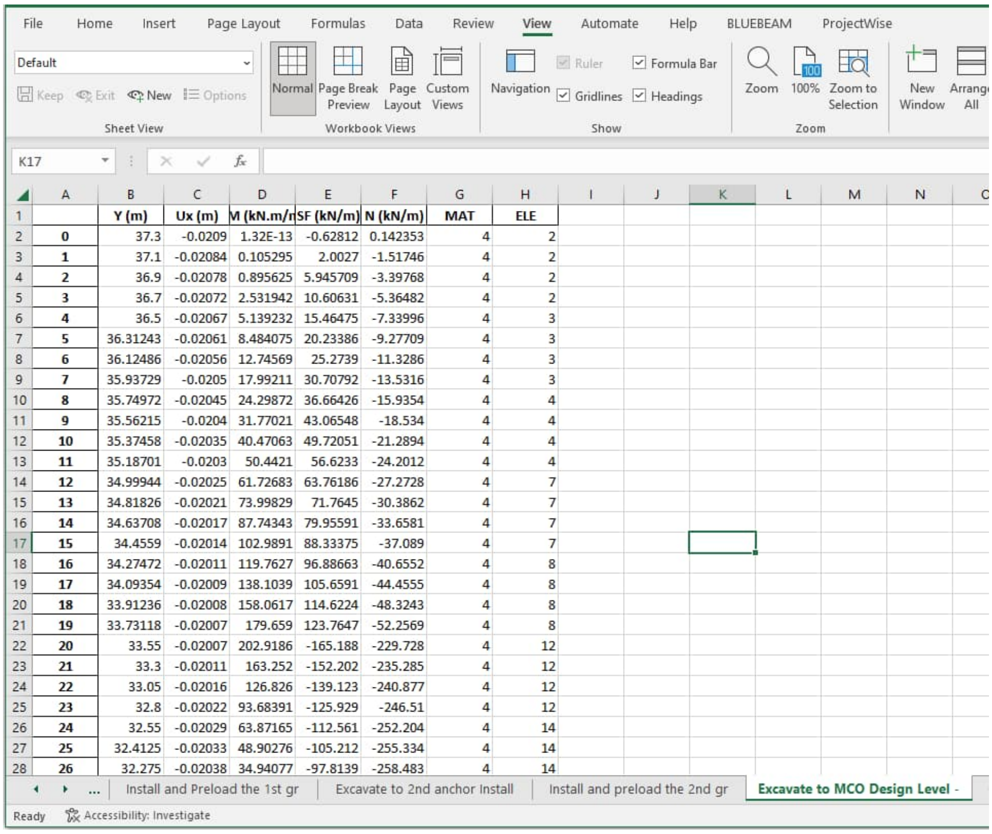


Figure 9: Typical PlaxisPlate2D output file – "Plots" sheet



	Y (m)	Ux (m)	M (kN.m/m)	SF (kN/m)	N (kN/m)	MAT	ELE
0	37.3	-0.0209	1.32E-13	-0.62812	0.142353	4	2
1	37.1	-0.02084	0.105295	2.0027	-1.51746	4	2
2	36.9	-0.02078	0.895625	5.945709	-3.39768	4	2
3	36.7	-0.02072	2.531942	10.60631	-5.36482	4	2
4	36.5	-0.02067	5.139232	15.46475	-7.33996	4	3
5	36.31243	-0.02061	8.484075	20.23386	-9.27709	4	3
6	36.12486	-0.02056	12.74569	25.2739	-11.3286	4	3
7	35.93729	-0.0205	17.99211	30.70792	-13.5316	4	3
8	35.74972	-0.02045	24.29872	36.66426	-15.9354	4	4
9	35.56215	-0.0204	31.77021	43.06548	-18.534	4	4
10	35.37458	-0.02035	40.47063	49.72051	-21.2894	4	4
11	35.18701	-0.0203	50.4421	56.6233	-24.2012	4	4
12	34.99944	-0.02025	61.72683	63.76186	-27.2728	4	7
13	34.81826	-0.02021	73.99829	71.7645	-30.3862	4	7
14	34.63708	-0.02017	87.74343	79.95591	-33.6581	4	7
15	34.4559	-0.02014	102.9891	88.33375	-37.089	4	7
16	34.27472	-0.02011	119.7627	96.88663	-40.6552	4	8
17	34.09354	-0.02009	138.1039	105.6591	-44.4555	4	8
18	33.91236	-0.02008	158.0617	114.6224	-48.3243	4	8
19	33.73118	-0.02007	179.659	123.7647	-52.2569	4	8
20	33.55	-0.02007	202.9186	-165.188	-229.728	4	12
21	33.3	-0.02011	163.252	-152.202	-235.285	4	12
22	33.05	-0.02016	126.826	-139.123	-240.877	4	12
23	32.8	-0.02022	93.68391	-125.929	-246.51	4	12
24	32.55	-0.02029	63.87165	-112.561	-252.204	4	14
25	32.4125	-0.02033	48.90276	-105.212	-255.334	4	14
26	32.275	-0.02038	34.94077	-97.8139	-258.483	4	14

Figure 10: PlaxisPlate2D output file – detailed results for individual analysis stage

Figure 10 shows the typical detailed results for an analysis stage which are used in the plots as shown in Figure 9.

Some limitations associated with this tool are summarised as below:

- Only vertical plate elements are supported by this post processing tool.
- Ground anchors modelled by node-to-node anchors shall start from the selected plate elements. The starting point of the node-to-node anchor shall share the same X coordinate with the plate.
- No Excel spreadsheet file will be created if no plate elements within the defined range are present.
- Only deflection, shear force, bending moment and axial forces for plates and axial forces for node-to-node anchors are extracted by PlaxisPlate2D. No other results are included.
- PlaxisPlate2D does not support the old version of Plaxis 2D. The users need to ensure that Plaxis 2D Version 22 or later is already installed on the computer before this tool can be installed.

5 CONCLUSIONS

A simple procedure based on the volume strain approach has been proposed to generate the high in-situ horizontal stresses within Class I/II rocks for the cut and cover tunnel analyses of M4 East project in 2015. Since then, this approach has been widely used for many infrastructures project including Sydney Metro City, Sydney Metro Northwest, Rozelle Interchange, M6, Western Sydney Airport (SBT) and Melbourne Metro. The exact linear relationship of the in-situ horizontal stresses can be simulated by this approach in PLAXIS 2D analyses for the cut and cover tunnels. There is no need to divide the rock layers into a number of sub-layers with different K_0 values to approximate the high in-situ horizontal stresses. It is a simple but more robust approach for the initial stress generation within the Class I/II rocks with high horizontal stresses. This approach also can be easily extended to model and rock having high horizontal stresses including Plaxis 3D modelling if required.

A Python based post processing tool has been developed for automatically extracting and presenting the analysis results from Plaxis 2D. This tool can achieve a great deal of time saving and avoiding the potential human errors in data processing. This tool has been successfully adopted on several major infrastructure projects in Australia, including Sydney Metro, Melbourne Metro, Western Sydney Airport (SBT), Sydney Metro West – Eastern Tunnelling Package, Suburban Rail Loop – Package C, and T2D Torrens to Darlington, consistently receiving very positive feedback. The tool can be made available upon request.

6 DISCLAIMERS

The authors, contributors and their respective organisations do not make any representation or warranty as to the accuracy, completeness, or suitability or otherwise of the information contained in this paper and shall have no liability to any person in connection therewith.

7 ACKNOWLEDGMENTS

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CRediT authorship contribution statement

Bo Xu: Conceptualisation, Software, Writing - original draft. **Qijing Yang:** Conceptualisation, Software, Writing - original draft, Writing – review and editing.

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REDEVELOPMENT OF HERITAGE BUILDINGS IN SYDNEY: CASE HISTORY OF COMPLEX UNDERPINNING AND SUPPORT WORKS

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ABSTRACT

The redevelopment of Sydney's Central Business District has prompted a trend toward updating heritage buildings to include modern functionality while preserving their historical integrity. The Lands Department ‘Sandstone’ project transforms a heritage sandstone building into a luxury hotel, requiring a deep shaft excavation within the heritage building footprint, followed by a mined tunnel to connect to the adjacent heritage which is also updated.

The project geotechnical challenges included numerous rock joints due to proximity to the G.P.O. Fault Zone, resulting in rock wedges and other mechanisms that required underpinning of the heritage and new foundations around the new shaft. The presence of sensitive heritage structures with limited deformation tolerance, coupled with restricted site access, further complicated design and construction. Site investigations demonstrated the variable geotechnical conditions, prompting downhole camera and trial trench inspections to confirm the rock jointing locations with precision for underpinning design.

A key structural challenge was to support an existing five-story sandstone block wall directly above the shaft. A precast concrete beam with integrated steel members was designed as a load-transfer structure, redistributing wall loads onto new underpinned foundations. The underpinning of the new transfer beam and the other existing footings alongside the proposed shaft excavation were supported by rock bolts. To address building settlement risks, flat jacks were incorporated into the transfer structure foundations with staged excavation and design verification. Through innovative engineering solutions, and meticulous planning construction and verification, this project successfully solved the geotechnical complexities associated with the redevelopment of the heritage building, and provides a valuable precedent for future similar developments.

1 INTRODUCTION

The redevelopment of Sydney's Central Business District has prompted a trend toward updating heritage buildings to include modern functionality while preserving their historical integrity. The Lands Department ‘Sandstone’ project transforms a heritage sandstone building (Figure 1:) into a luxury hotel, requiring a deep shaft excavation within the heritage building footprint, followed by a mined tunnel to connect to the adjacent heritage building which is also updated.

Figure 1: Photograph of existing heritage sandstone buildings being redeveloped

Figure 2: Site locality

The site is located in the northern part of Sydney's CBD close to Circular Quay (Figure 2:), on a block bounded by Bridge, Loftus, Bent, and Gresham Streets. The four-storey sandstone building was built around 1880 and historically was occupied by the NSW Department of Lands. Until as recently as 2016 it was occupied by the NSW Department of Planning.

Some notable historical features of Lands Department building comprise:

- The Victorian-style building façades, crafted from sandstone blocks and adorned with classical sculptures and arched windows
- A significant feature on the Bridge Street façade is the Lands Department Datum Benchmark Plug, which served as the origin for all survey levels in New South Wales
- The central clocktower rising to a height of 5 storeys
- Moat walls surrounding the perimeter of the building basement on Gresham, Bridge and Loftus Street to provide essential light and ventilation to the basement

2 SITE CONSTRAINTS

The redevelopment of the building into a luxury hotel required a 10 m deep shaft to be excavated within the existing building footprint. This shaft was to provide "back of house" functionality to the hotel, as well as access to the new tunnel underneath Loftus Street that connects into the adjacent redeveloped heritage building. A series of precast slabs are to be placed over top of the shaft to provide continuity of the hotel ground floor.

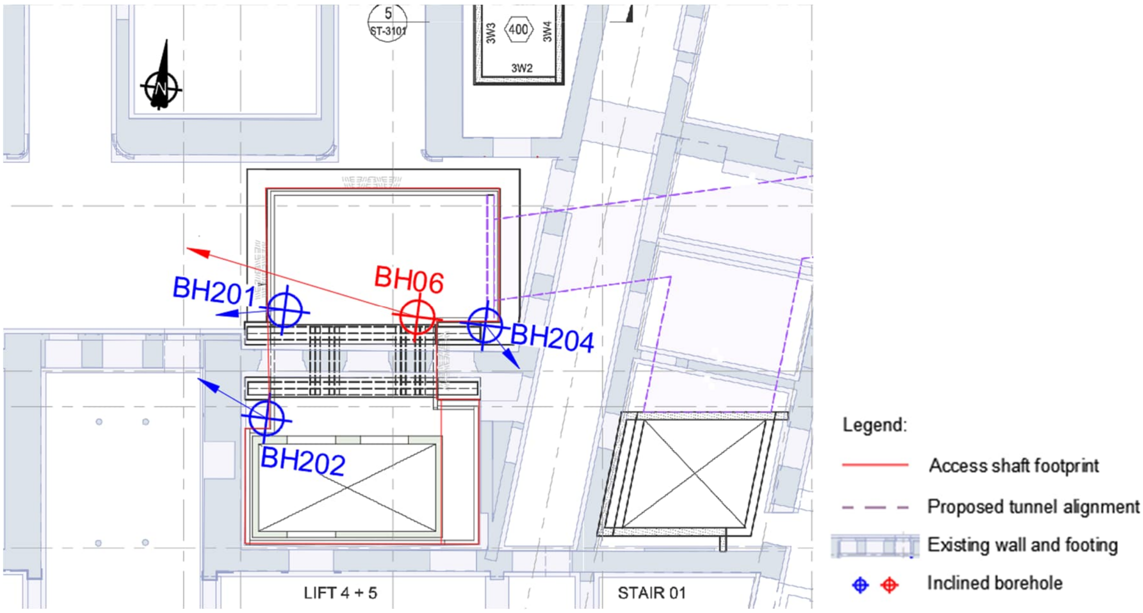


Figure 3: Site plan

Physical constraints to the shaft include:

- An existing four storey / five-level sandstone block wall with window openings, spanning directly over the shaft location.
- Other sandstone block walls surrounding all four sides of the shaft; the bearing pressures beneath these walls are in the order of 500 kPa (i.e. due to the weight of up to 25 m height of stacked sandstone blocks!).
- Adjacent to the western side of the shaft, a clocktower that extends above the top of the building (Figure 1:) and imparts a bearing pressure of over 1000 kPa.
- The above-mentioned walls and clocktower are all founded on one to two rows of unconnected roughly (hand) cut sandstone blocks in the order of 1 m long and 0.5 m wide and high, stacked on top of each other and founded on the underlying insitu bedrock, with nearly zero offset to the excavation face (Figure 3:).
- The new shaft ceiling slabs also impose loads around the edge of the shaft due to the self weight of the concrete and also the soil and trees of the planter boxes contained on the slabs.

Each of the heritage elements adjacent to the shaft had tight limits on total and differential movements, particularly given their construction from sandstone blocks.

Site constraints for construction meant that there was very limited access to machineries and most of the work had to be undertaken using small plant.

3 GEOTECHNICAL MODEL

3.1 GEOLOGICAL SETTING

Sydney CBD is underlain by Hawkesbury Sandstone (Pells et al, 2019), which is typically a medium to high strength horizontally bedded sedimentary rock unit with either massive or cross-bedded fabric. At this site, the main geological feature of relevance is the “G.P.O. Fault Zone” which is a zone where sub-vertical joints striking NE-SW are expected to be encountered more frequently (Figure 4:). It is important to note that this does not mean that the entire width of the G.P.O. Fault Zone as mapped (Och et al (2004), Pells et al (2004)) contains closely spaced joints. Likewise, it also does not mean that conditions return to “normal” immediately beyond the extent of the G.P.O. Fault Zone as mapped, simply that the prevalence of sub-vertical joints will gradually decrease.

At this site and around the Sydney CBD there is also extensive prior land use resulting in variable depth and character of fill. At this site, there were historic excavations for pipes, footings, pits, etc that either contained fill or were simply voids full of water. However the natural soil profile had been essentially removed, meaning he fill directly overlay the sandstone bedrock.

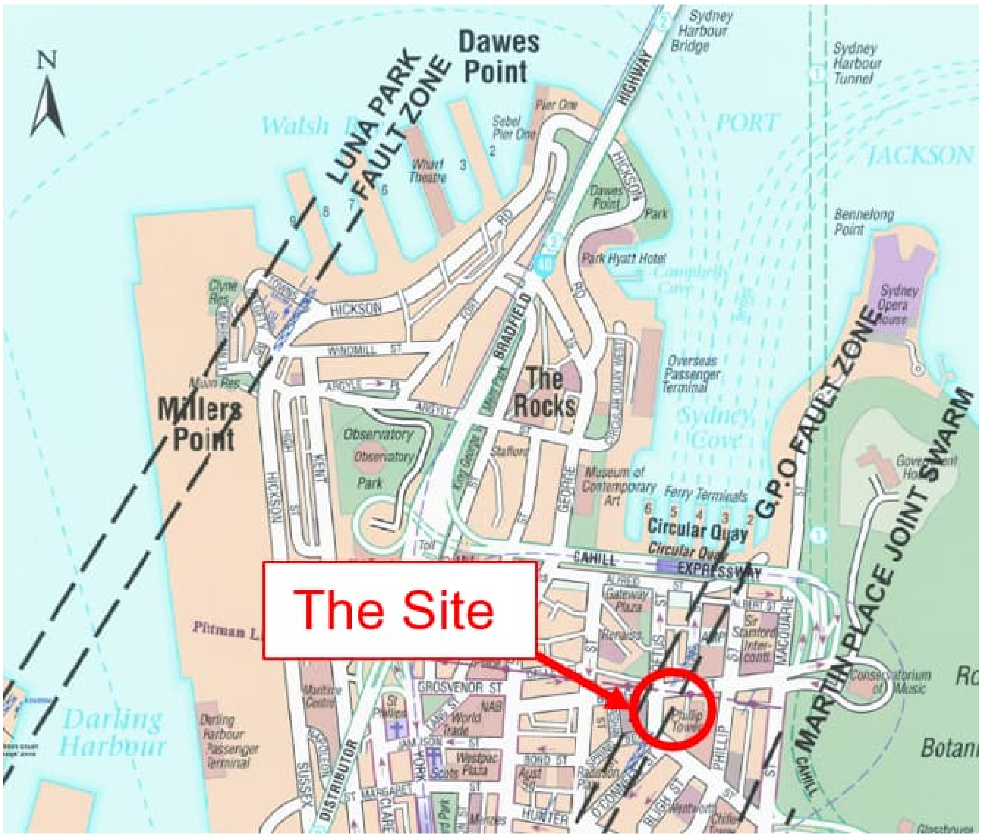


Figure 4: Geological setting (from Och et al 2004 and Pells et al 2004)

3.2 SITE INVESTIGATION

Based on the expectations of the geological setting, a series of inclined boreholes (Figure 3:), ranging from 5 to 7.2 m at 70° inclination angle, were undertaken, particularly to assess the presence of NE-SW striking defects in areas of most relevance to the design. Downhole camera inspections (Figure 5:) were undertaken within the boreholes to provide further clarity on the insitu condition of the defects.

The rock mass encountered consisted of typical medium strength sandstone with some localised low strength rock occasionally interbedded with the rock mass. A 20 to 40 mm thick, sub-horizontal clay seam was observed intersecting the deep boreholes. This clay seam separated two distinct lithologies. The upper sandstone unit was less weathered and of low to medium strength, while the underlying unit was moderately weathered with iron staining and was of medium to high strength.

Downhole camera inspections using an endoscope camera identified two joint sets within the cored boreholes. The first set was sub-vertical with a strike direction of NNE to NE, consistent with the orientation of the nearby GPO fault zone. These joints were closely spaced, with an average spacing of 0.1 to 0.2 m. The second joint set was also steeply dipping, striking NW-SE and widely spaced.

During the detailed design, it was collectively decided between the geotechnical engineer and contractor to undertake trial trenches within the shaft near its perimeter. The main purpose of the trial trenches was to further increase the certainty of the precise location of defects prior to construction, such that the design could be almost completely prescriptive and not have to respond to changes in conditions encountered during construction. The trial trenches, excavated approximately 1 m deep into sandstone bedrock (Figure 6:), confirmed the persistence and orientation of the NNE-NE striking joint set on both sides of the sandstone block wall to be underpinned. The projection of this joint set indicated a potential intersection with the eastern rock pillar designated to support the new transfer beam footing and extending behind the tunnel portal face. Every defect intersecting the top part of the shaft wall was encountered and precisely mapped, to inform the design.

The outcome was an essentially deterministic geotechnical model (Figure 7:) which was used to show the locations of all of the defects with respect to the proposed underpinning and ground support works. Most of the sub-vertical defects were concentrated within two distinct zones that traversed the shaft and underlay the proposed rock pillars for the underpinning.

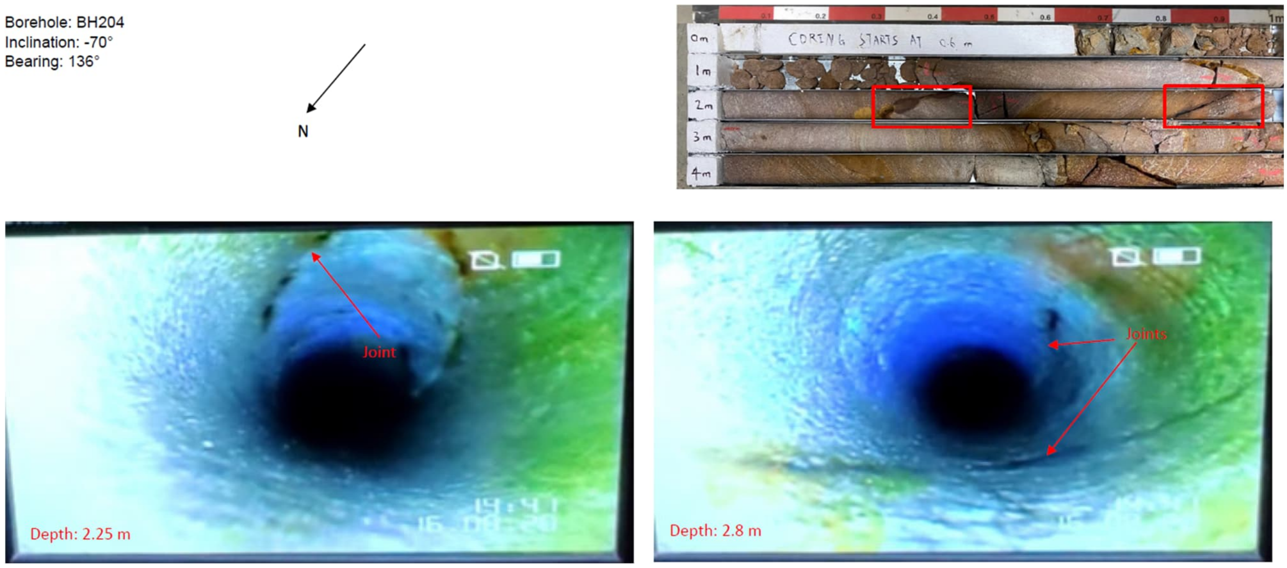


Figure 5: Typical borehole conditions – core photo (top) and endoscope inspection records (bottom)

Figure 6: Typical jointing observed in trial trenches, refer to the blue hatched zone in Figure 7

3.3 IMPLICATIONS FOR DESIGN AND CONSTRUCTION

With the site investigations having confirmed the geotechnical conditions, the design implications of the presence of the sub-vertical jointing within the Hawkesbury Sandstone included:

- Requirement to provide support to potential rock wedges, unconfined compressive mechanisms within the rock substance, and other mechanisms beneath the heritage walls around the shaft perimeter.
- Requirement to provide support to potential large-scale rock wedges and/or planar sliding and/or toppling mechanisms occurring within the narrow rock pillars underneath the underpinning pad footings.
- Requirement to provide support to potential rock wedges above the tunnel crown.
- Consideration of the effects of excavation-induced stress relief, although the shaft was highly three-dimensional which was expected to result in minimal ground deformations.

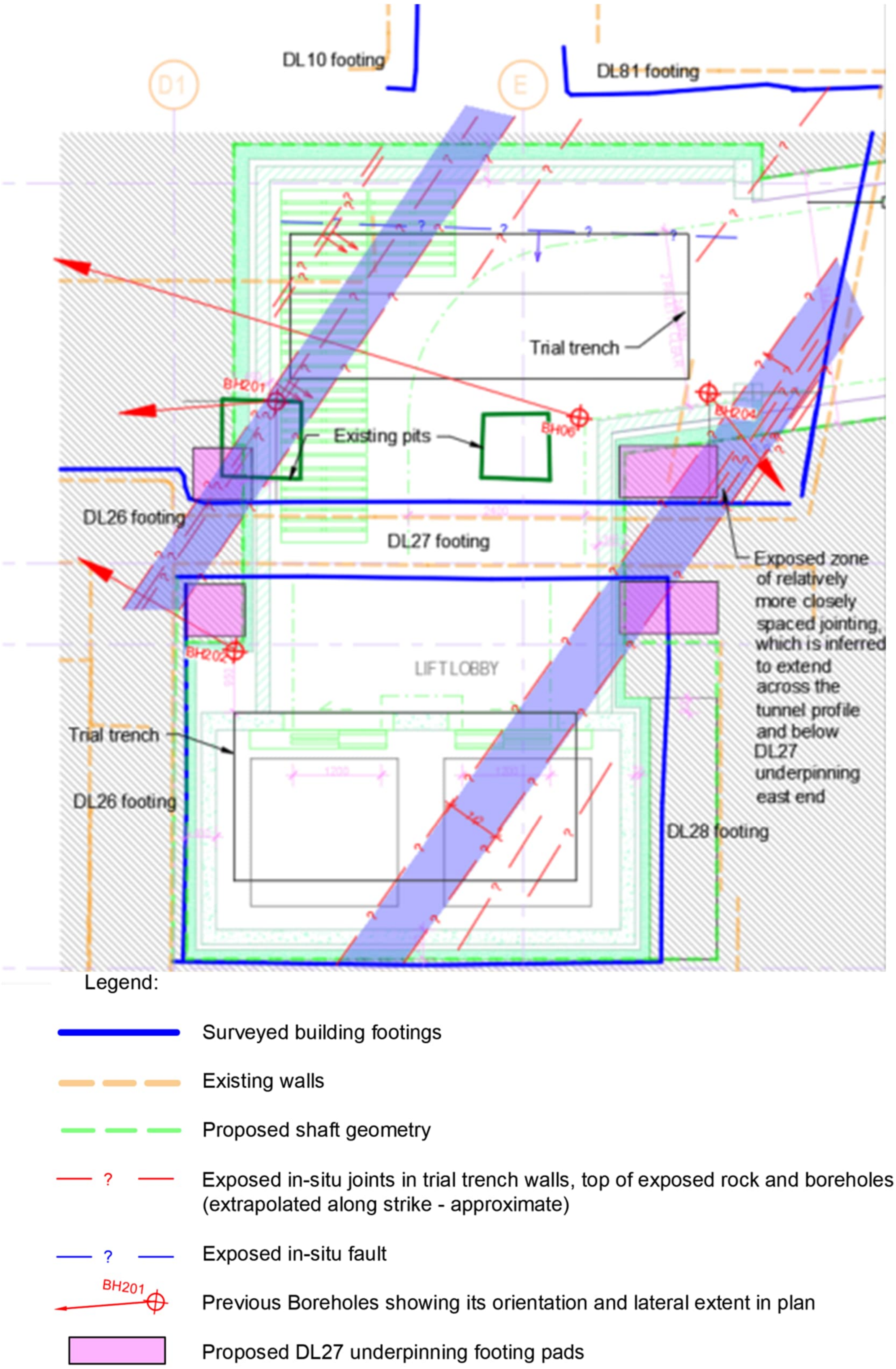


Figure 7: Geotechnical model plan

4 DESIGN SOLUTION

4.1 TRANSFER STRUCTURE

The structural solution for transferring the load of a five-storey building wall arching over the proposed shaft excavation consisted of a pair of prefabricated steel underpinning beams (7 m long 940x400 custom steel I beams encased in cast in situ concrete), positioned longitudinally on either side of the existing wall toe, integrated with the transverse mini-niche beams (310 UC 158) through the bedrock immediately beneath the sandstone block footings. These elements were designed to span across and be found on rock beyond the shaft footprint as a load-transfer structure, ultimately redistributing wall loads onto new underpinned foundations. The structural details were designed by the structural engineer, refer to Figure 8: for the typical cross-section of the mini niche beam to connect between longitudinal steel members on each side of the existing wall.

The transfer structure was required to control ground deflection of under span/300 to reduce the cracks to the heritage wall, along with the subsequent excavation. To address building settlement risks, flat jacks were incorporated into the transfer structure foundations to offset the anticipated building settlement associated with the shaft excavation and transfer of loads via the underpinning beams onto the new footings and rock pillars. The integrated jacking system was designed to accommodate up to 25mm vertical and 15mm lateral convergence as a result of the relief of stress concentrations in the rock mass, known as rock mass relaxation. In Sydney CBD, the lateral movements in rock at the surface around the excavation perimeter have usually ranged between 0.5mm and 1mm per meter depth of rock excavation (Pells (1990)). It was anticipated that the stress relief movements from the shaft excavation would be at the lower end of precedent (i.e. no greater than 0.5 mm per metre), given its proximity to the ground surface and its relatively small excavation footprint in plan, creating a three-dimensional effect that helps resist rock relaxation.

The construction sequence and support strategy are crucial to the success of load transfer under the strict ground deformation limit. The geotechnical and structural engineers worked collaboratively with the contractor and the structural designer to integrate the constructability, structural and geotechnical capacity. Refer to Section 5 – Construction Sequence for further details.

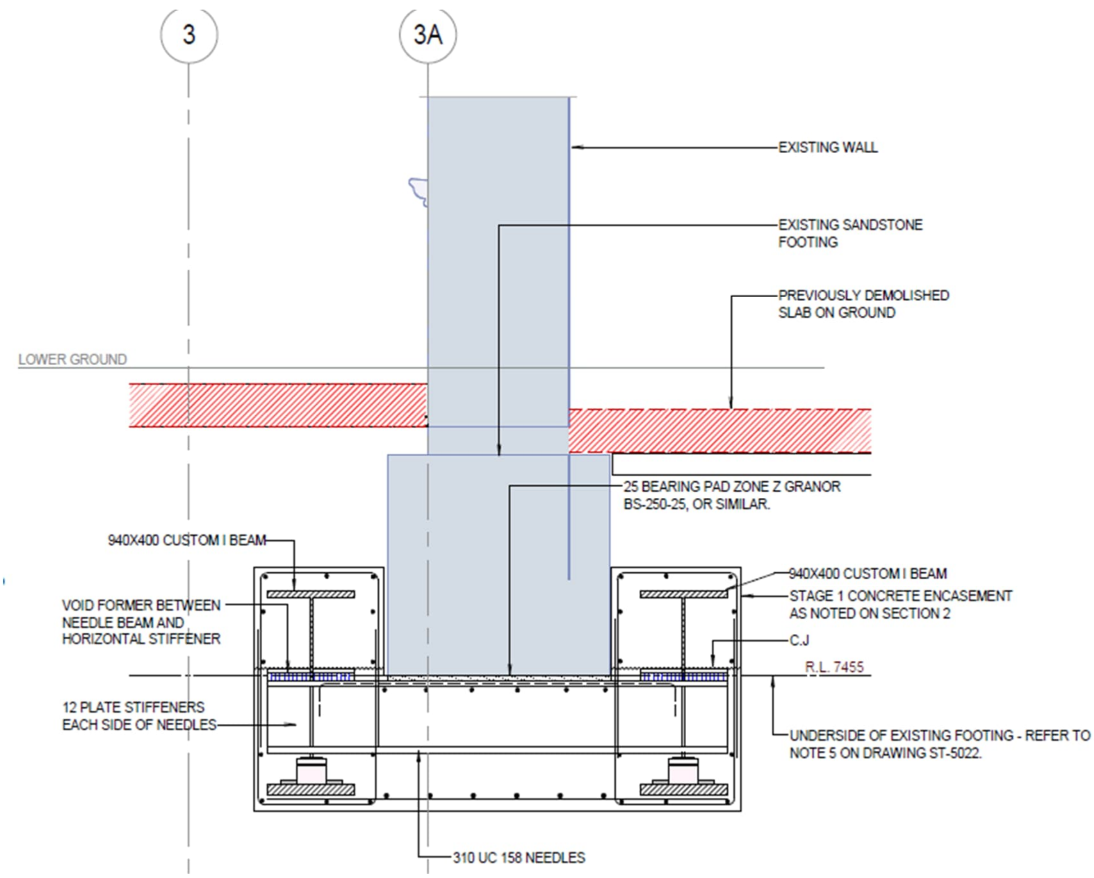


Figure 8: Typical cross section showing the connection between the mini niche beam and longitudinal steel members

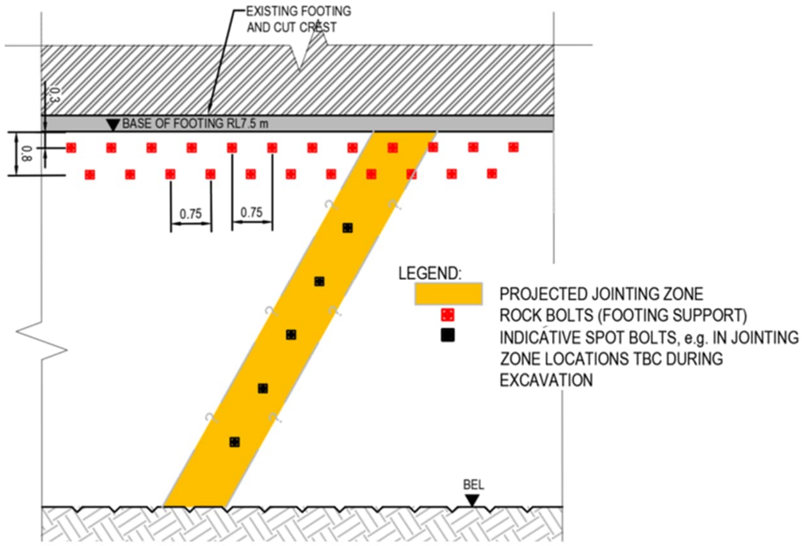
4.2 UNDERPINNING

Due to the potential for various failure mechanisms of the rock face as surcharged by the footings of the existing walls, including planar, wedge, toppling, and rock mass mechanisms, underpinning was required for both the existing walls and of the more highly loaded new transfer structure footings. Typical loads of the existing walls ranged from 380 to 1670 kN/m, whereas the loading on the new transfer structure footings was 720 kPa. The underpinning system consisted of multiple rows of Glass Fibre Reinforced Plastic (GRP) bolts, 40 mm diameter solid bars with bolt lengths ranging from 3.5 to 6 m. The bolts were designed to secure against potential instability under the applied footing loads, including:

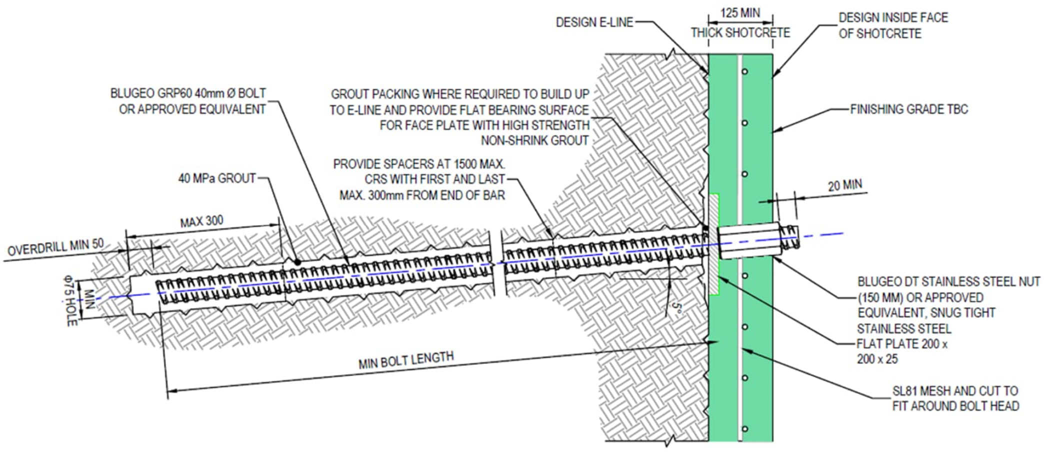
- Planar sliding along the moderately to steeply dipping NE-SW striking joints as a basal plane daylighting above the wall toe,
- Two-sided wedges formed by the intersection of two distinct and oblique joint sets daylighting at the cut face (e.g. NE-SW and NW-SE striking joints), and
- Toppling failures occurred at the crest of the cut face induced by the sub-vertical joints dipping into the wall.

The design was governed by the rock bolt pull-out capacity, defined as the minimum of three values: the grout-to-rock bond strength, the bolt-to-grout bond strength, and the tensile capacity of the GRP bar. See Figure 9: for the typical pattern bolt layout and typical shotcrete and GRP bolt details to underpin the existing wall footings adjacent to the shaft footprint.

For the rock pillars beneath the new transfer beam, a specific pattern of rock bolts was designed to be installed from both sides of the pillar. This bolt arrangement was designed to restrain the largest potential planar mechanisms and wedges identified as being formed by the persistent NE-SW striking joint swarm (see Figure 10: for configuration details), as well as smaller wedges and toppling mechanisms.



a)



b)

Figure 9: (a) Typical underpinning bolt layout and (b) Typical shotcrete and GRP bolt details

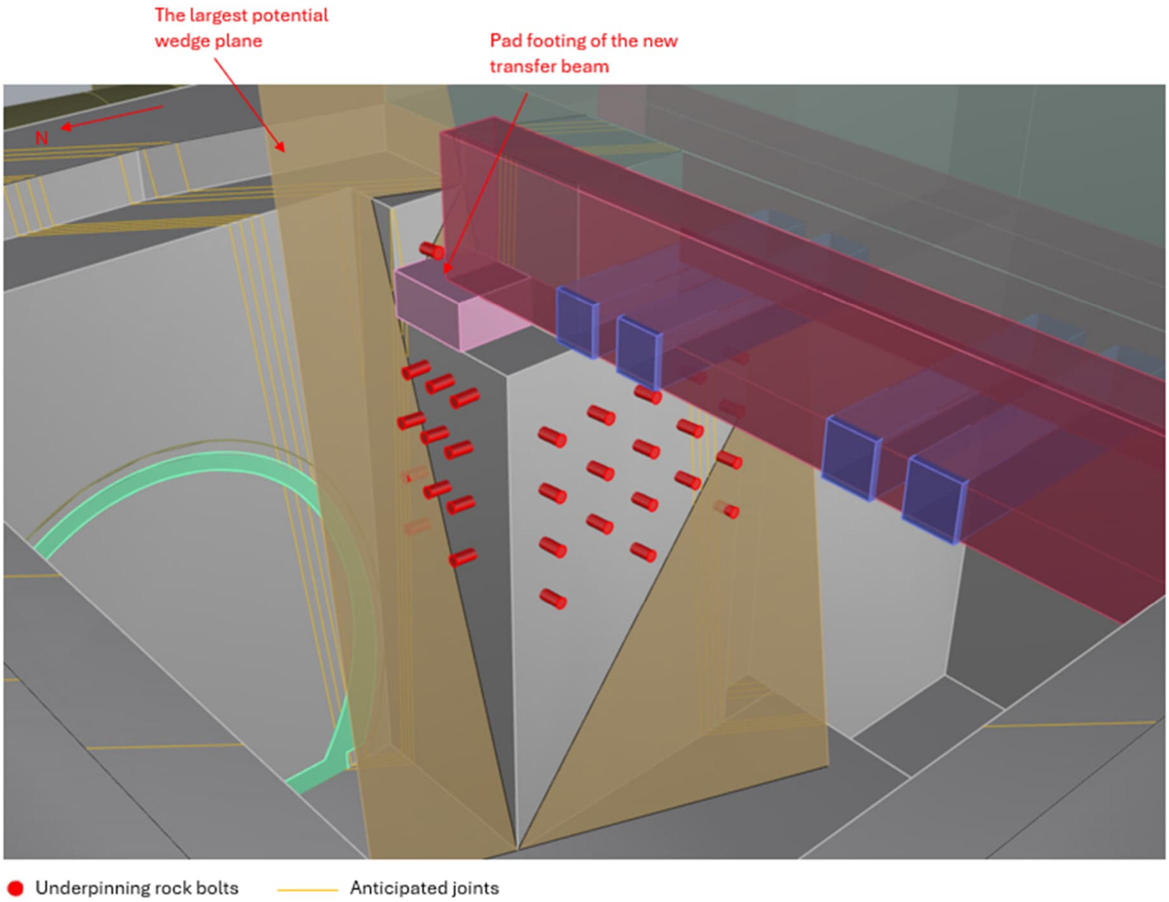


Figure 10: Underpinning design model (NB – 3D image of the pillars, jointing, and bolts)

The shaft walls were finished with a shotcrete facing for architectural purposes, and this also met the 100-year design life requirement. This high-performance finish was also specified to meet the durability requirements for the hotel basement. Although no significant groundwater ingress was anticipated, regularly spaced strip drains were installed behind the shotcrete, which was designed to capture any trapped water and divert it to a cavity drain at the toe of the wall, preventing hydrostatic pressure buildup behind the shotcrete wall.

4.3 TUNNEL PORTAL

The integration of the shaft and tunnel portal designs and construction sequence was critical for maintaining face stability during the tunnel excavation beneath existing building footings. The design approach was to provide permanent face support for the shaft wall as well as temporary support for the potential planar wedges anticipated to develop above the tunnel crown prior to installing the permanent tunnel lining.

Site investigation indicated ground conditions at the north-south striking tunnel portal comprised Sandstone Class III overlying Sandstone Class I/II with closely spaced sub-vertical joints striking NE-SW identified behind the tunnel portal. These adversely oriented joints were considered kinematically critical to the tunnel portal, potentially forming the basal plane of large scale planar wedges underneath the existing building footings, especially upon the tunnel breakthrough, which would undermine the wedge toe (see Figure 11:). The underpinning system consisted of multiple rows of double corrosion protection (DCP) steel bolts, 24 mm diameter solid bars with bolt length of 4 m, installed in a 75 mm diameter borehole. The bolts were installed beyond the basal planes and at a slightly inclined bolt angle to increase the resisting force component along the basal planes.

The design philosophy considered both a best guess and a worst-case scenario, aiming to support the largest kinematically feasible wedge defined by geometrical constraints. The design solution featured four rows of inclined double corrosion protection rock bolts installed at the portal face and embedded through the anticipated back release plane, designed as a cantilever system. The group of rock bolts were designed to provide adequate net pull-out capacity and shear strength to support the weight of the planar wedge and its associated footing loads. In addition, given the limited deformation tolerance underneath the existing footing, a three-dimensional (3D) numerical analysis was performed to assess the potential deformation of the wedge and overlying footings during staged tunnel excavation. The calculated footing movement was within the design tolerance.

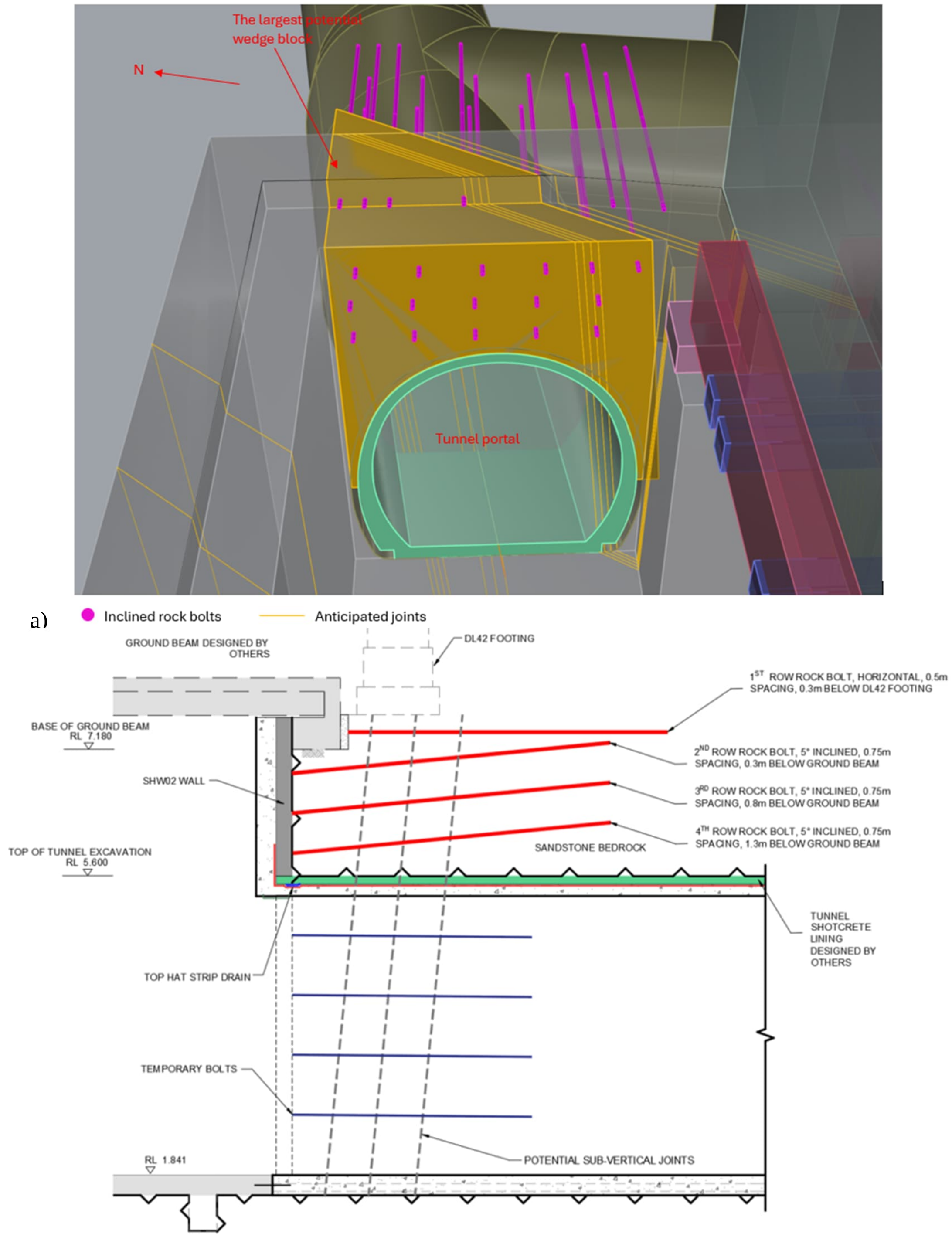


Figure 11: (a) Tunnel portal design model (NB – 3D image of the portal, jointing, and bolts) and (b) bolt layout along the tunnel profile

During shaft excavation, further verification of the tunnel portal ground model was undertaken to reduce the uncertainty regarding potential planar wedges. Borehole camera inspections within selected rock bolt drillholes were conducted to observe the presence of in-situ joints behind the shaft walls and confirm the extent of the potential wedge. It was confirmed that the wedge basal plane striking NE-SW behind the tunnel portal was present and consistent with the geological models adopted in the design, however, the anticipated east-west striking defect set, crucial for acting as a side release plane, was not positively identified. The absence of this side release plane implied that breakout through the sandstone rock mass between the tunnel portal and the basal plane would be necessary to form a planar wedge. This significantly reduced the assessed likelihood of wedge instability and consequently, reduced the minimum support required on the shaft wall.

5 CONSTRUCTION SEQUENCE

The construction sequence for the access shaft, particularly the installation of a transfer beam to support existing building structures while minimising ground deformation during subsequent bulk excavation, demanded multidisciplinary integration (constructability, structural and geotechnical). The sequence adopted for the access shaft excavation was:

1. Early Works
   a. Localised exposure and inspection of existing wall footings adjacent to the proposed shaft excavation to verify the as-built geometry and condition of the sandstone block footings and the nature of the underlying foundation materials.
   b. Excavation of trial pits within the shaft footprint to further investigate the extent and characteristics of the identified sub-vertical jointing prior to bulk excavation.
   c. Assessment of the temporary bearing capacity of existing footings that would be exposed during underpinning, informing the staged excavation and support strategy.
   d. Installation of monitoring prisms on existing structures and acquisition of baseline readings prior to commencing bulk excavation.

2. Transfer Structure Installation and Underpinning
   a. Temporary bracing of existing walls and stitching of the sandstone block footings to maintain their integrity during staged excavation.
   b. Localised box cuts at both ends of the existing wall to facilitate the installation of reinforced concrete pad footings for the proposed transfer beam.
   c. Controlled excavation of two parallel mini trenches within the sandstone bedrock immediately adjacent to and extending beneath the sandstone block footings, see Figure 12:.
   d. Installation of paired prefabricated steel members, positioned longitudinally alongside either side of the existing wall toe, followed by the insertion of transverse mini-niche beams through the openings created in the mini trenches to structurally connect these longitudinal steel members beneath the wall.
   e. Installation of flat jacks at the base of the mini niche beams and the transfer beam pad footings to actively control ground movement and pre-load the system (i.e. achieve a new load path for the wall into the ground via the new footings), thereby mitigating movement of the existing wall during subsequent excavation, see Figure 13:.
   f. Installation of specific underpinning (e.g. rock bolts, shotcrete) for the sandstone rock pillars supporting the transfer beam pad footing, undertaken prior to breakthrough excavation beneath the completed transfer beam.
   g. Staged breakthrough excavation beneath the completed transfer beam with real-time ground deformation monitoring and adjustment of flat jacks as required to control the ground movement
   h. Concrete encasement for the transfer structure upon verification of negligible ground deformation from the ongoing monitoring data following the shaft excavation to half of its planned depth, see Figure 14:.

3. Main Shaft Bulk Excavation (Post Transfer Beam Installation)
   a. Staged excavation of the remaining shaft on both sides of the completed transfer beam. Perimeter saw-cutting was employed along excavation boundaries adjacent to sensitive footing structures to minimise ground vibrations and disturbance to existing footings.
   b. Strict control of excavation lift heights (e.g. typical 1.0m lifts in the upper portions of the shaft), followed by immediate geotechnical inspection and mapping of the exposed cut face after each lift to verify the encountered ground conditions and adjust ground support if necessary.
   c. Systematic installation of ground support (e.g. rock bolts and shotcrete) to ensure the stability of the exposed sandstone rock face.
   d. The sequence of staged excavation, face inspection and support installation was repeated iteratively until the finished bulk excavation level was reached, see Figure 16:.

4. Tunnel Portal Works
   a. Implementation of a similar sequence to the main shaft bulk excavation.
   b. In addition, targeted borehole camera inspections were conducted within pre-drilled rock bolt holes at the designated portal location to further investigate in-situ jointing and confirm the geometry of potential planar wedges, supplementing earlier site investigation data.

c. The designed pattern bolting of four rows of inclined rock bolts was installed to stabilise anticipated planar wedges at the tunnel portal. Field adjustments to rock bolt locations were made where necessary to avoid clashes with pre-existing in-ground service trenches.
d. Upon completion of shaft excavation, careful staged excavation and installation of the primary tunnel support for the initial tunnel section commenced beneath the existing footings.
e. Subsequent works included the installation of permanent secondary lining and integration of structural connections and drainage systems between the shaft walls and the tunnel lining.

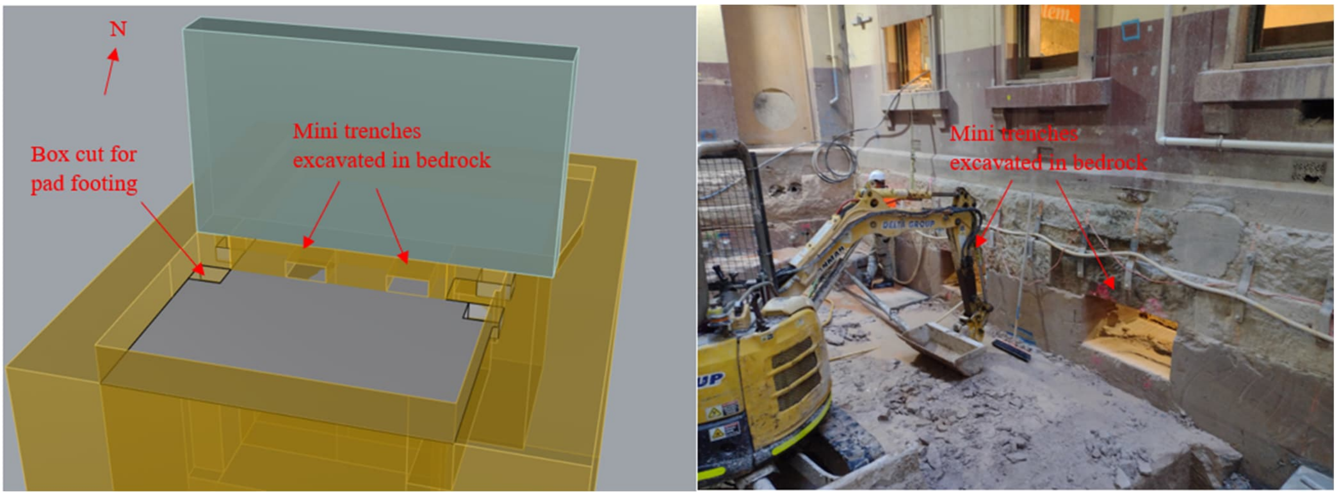


Figure 12: Pad footing and mini niche beam installation – 3D model isometric view (left) and site photography (right)

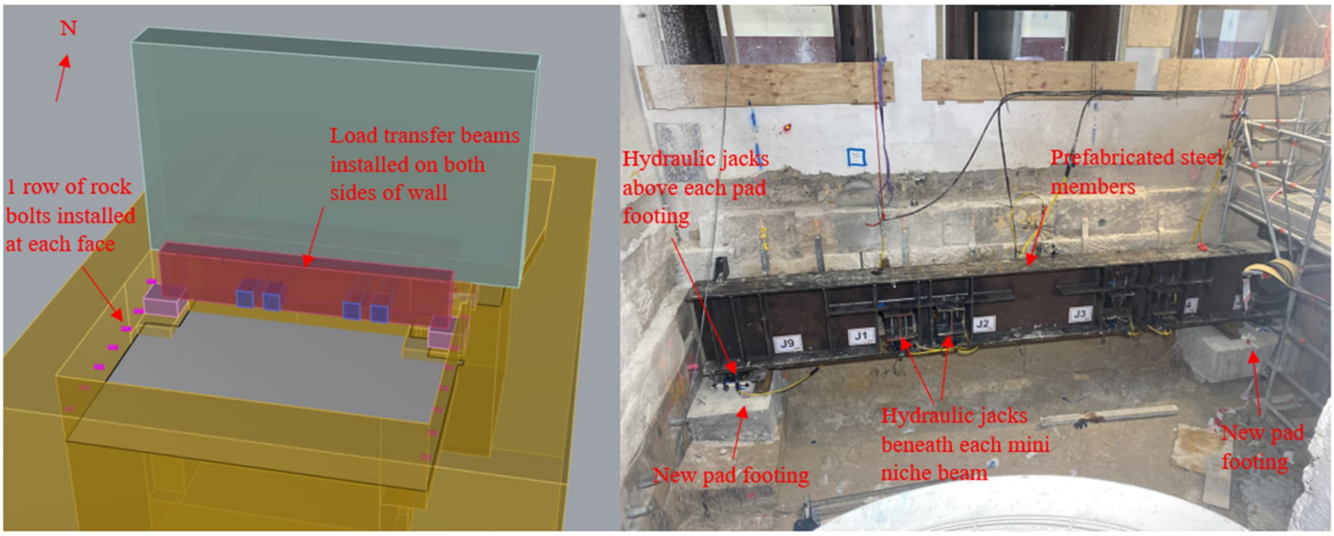


Figure 13: Load transfer beam and jacking system installation – 3D model isometric view (left) and site photography (right)

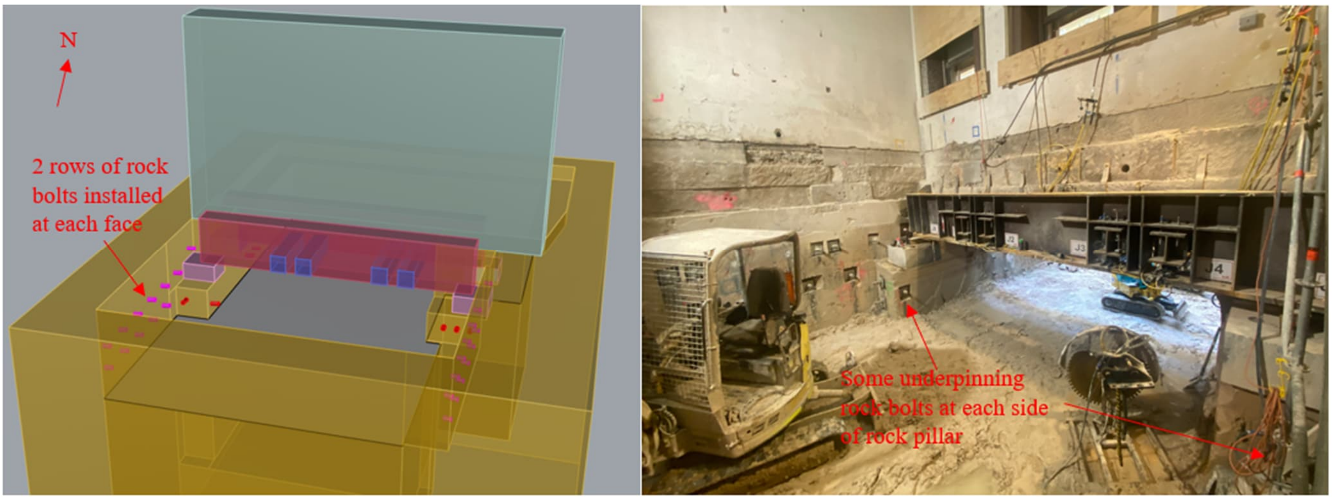


Figure 14: Wall breakthrough – 3D model isometric view (left) and site photography (right)

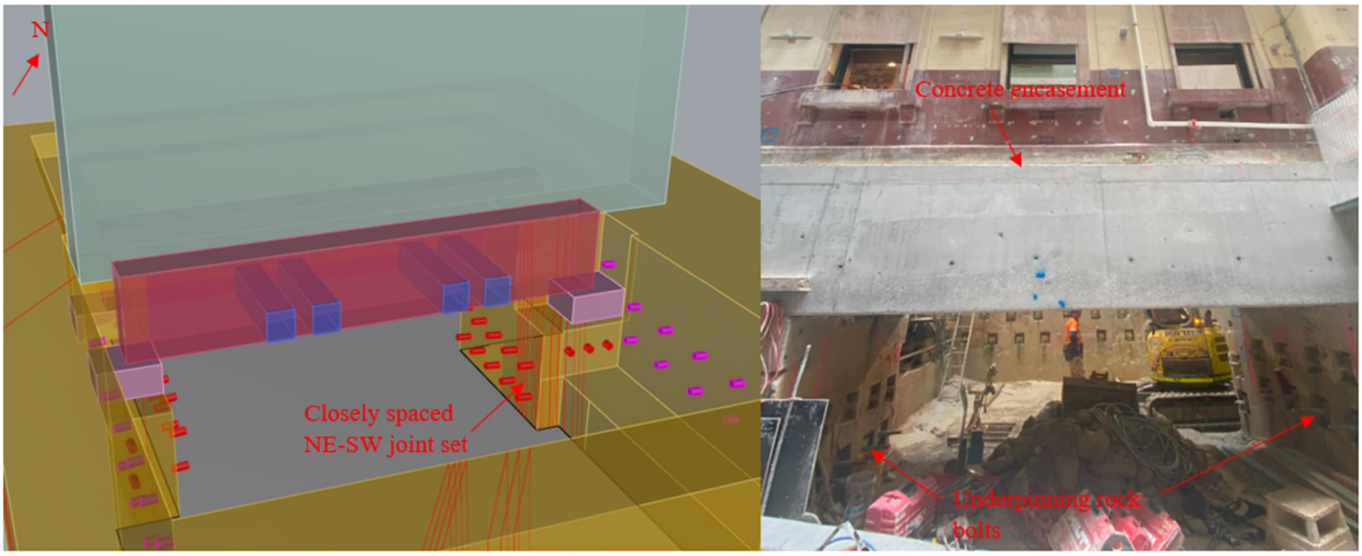


Figure 15: Underpinning the rock pillars - 3D model isometric view (left) and site photography (right)

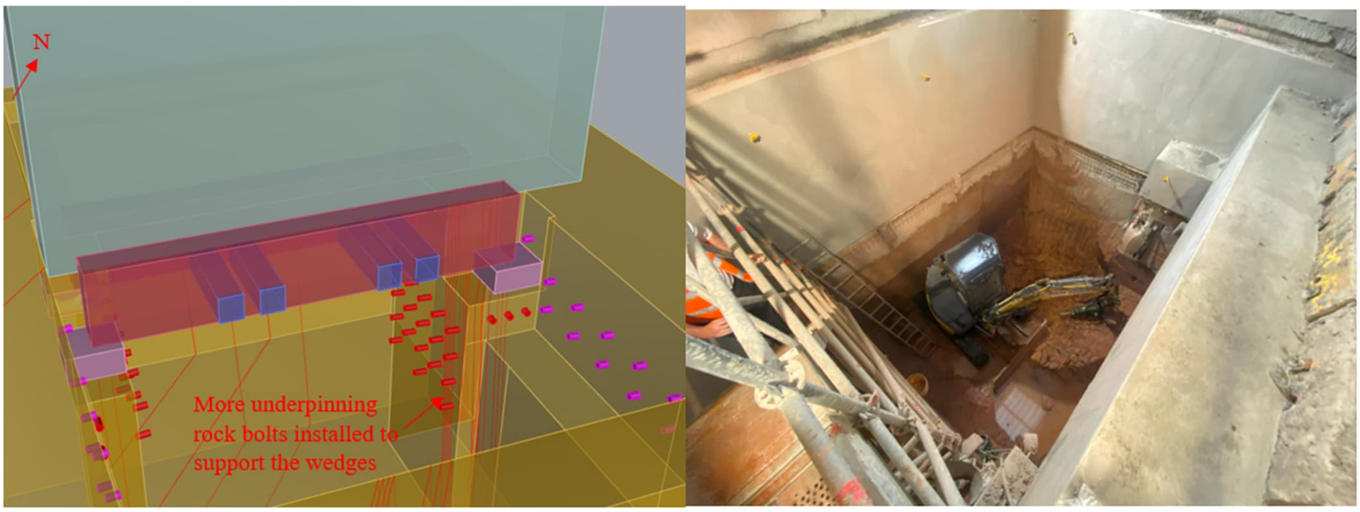


Figure 16: Bulk excavation of the remainder of access shaft - 3D model isometric view (left) and site photography (right)

6 CONSTRUCTION AND MONITORING

The successful execution of the design required a highly controlled construction approach, integrating specialised excavation techniques with a rigorous monitoring and design verification program. This was essential to manage the significant geotechnical risks and adhere to the strict deformation limits of the heritage structure. Key elements of this approach included:

- Close geotechnical involvement during construction including design verification such as hold points and witness points, and working collaboratively with the contractor to address issues and optimise the sequence.
- Saw cutting and line drilling, as opposed to rock hammering, to avoid overbreak and minimise vibrations.
- Rock bolt pull testing conducted to verify the design bond strength
- Maximum excavation lift heights specified, being 1 to 1.5 m at the top of the shaft and increasing to 2.5 m at the bottom of the shaft were rock mass conditions improved and the works were further from the heritage walls.
- Real-time wall monitoring.
- Adjustment of flat jacks as required to control the ground movement beneath the underpinned wall.

The implementation of these controlled construction and monitoring strategies proved highly effective. The measured movements of the existing walls and footings were negligible and remained well within the strict design tolerances throughout the construction. As a result, the impact of the access shaft excavation on the integrity of the existing heritage building was minimal. Figure 17: and Figure 18: show some photographs taken during the construction works.

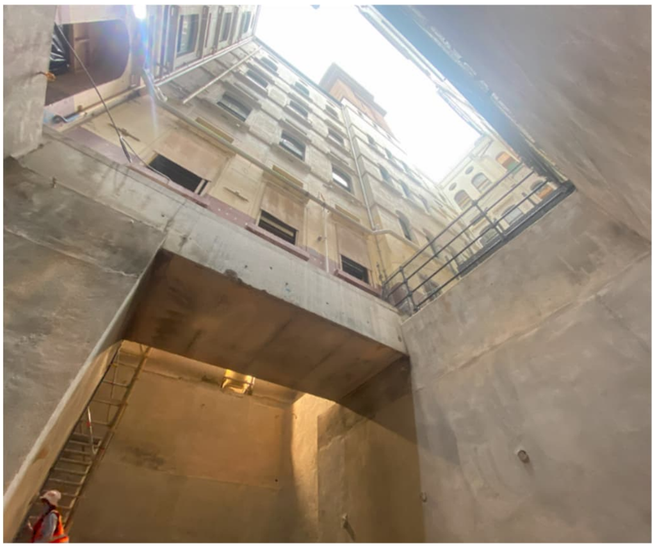


Figure 17: Photograph of the completed transfer structure, taken from the bottom of the shaft

Figure 18: Photograph of the completed tunnel portal before tunnel breakthrough (left) and after tunnel breakthrough (right)

7 CONCLUSIONS

The project case study presented in this paper is one example of an increasing trend for redevelopment of heritage buildings. Such redevelopments are often highly constrained by requirements to preserve and protect heritage elements. This particular project was additionally constrained by the geotechnical conditions namely the G.P.O. Fault Zone in Sydney's CBD.

Key project challenges included support of an existing five-story sandstone block wall directly above one of two new shafts, as well as support of similar walls and a higher clock tower around the perimeter of the shafts. Addressing these challenges required close integration between construction, structural, and geotechnical engineering disciplines.

Through innovative engineering solutions, and meticulous planning construction and verification, this project successfully solved the geotechnical and construction complexities associated with the redevelopment of the heritage building, and provides a valuable precedent for future development.

8 ACKNOWLEDGEMENTS

The authors acknowledge the construction expertise of Built and the structural expertise of TTW, in delivering this project.

This project was first presented to the AGS during an oral presentation by Mr Liang at the 2024 Sydney Chapter Young Geotechnical Professionals event, where it received the first prize award. This paper documents the presentation and provides further details. Thanks are extended to the organising committee for the opportunity to publish the paper in *Australian Geomechanics.*

CRediT authorship contribution statement

Juno Liang: Writing - original draft. **Jeremy Toh:** Writing – original draft.

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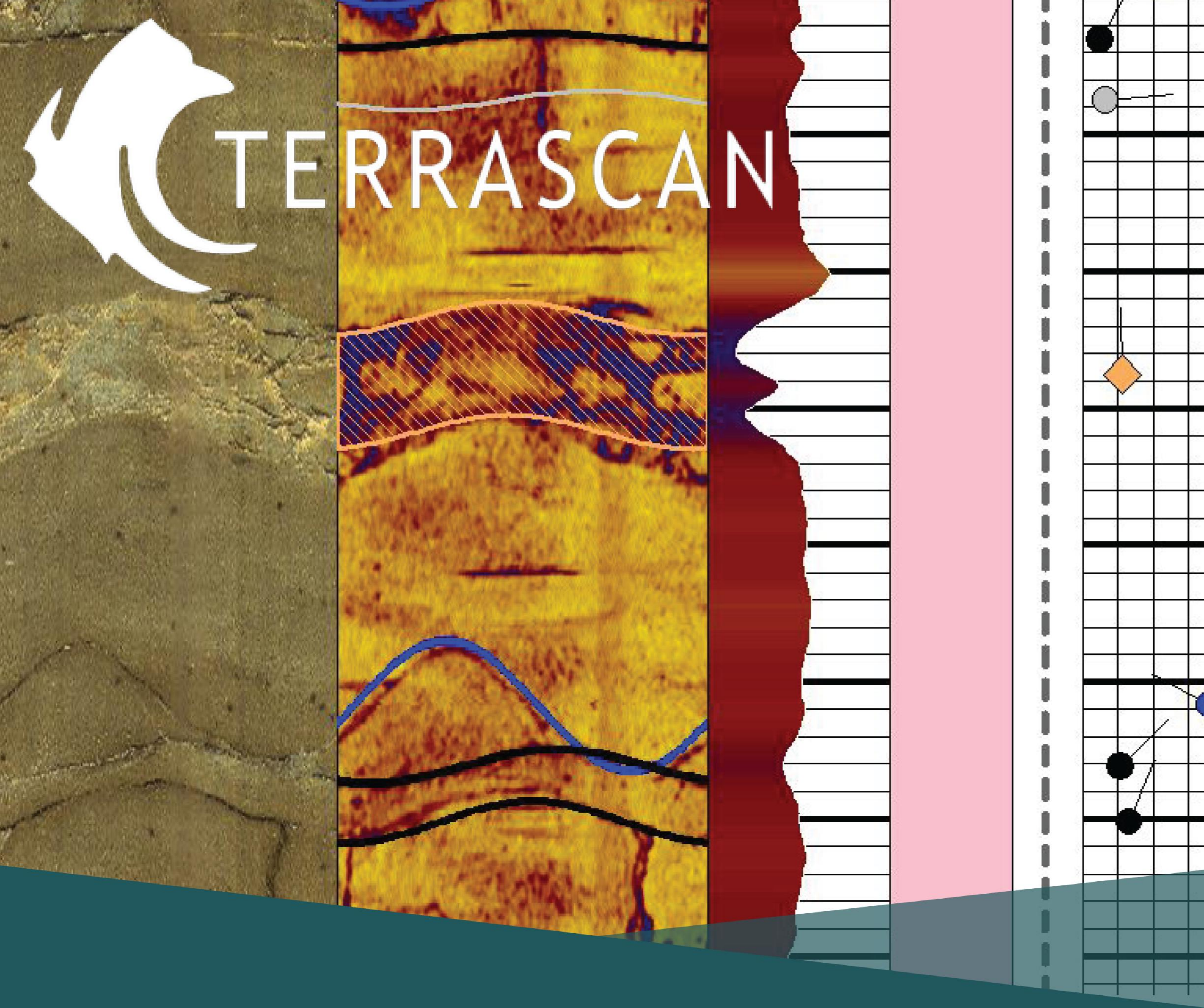
TERRASCAN

SOFT SOIL STABILIZATION USING ADMIXTURES FROM VARIOUS SOLID WASTE MATERIALS

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ABSTRACT

Foundations for civil infrastructure built on soft and compressible soils are prone to failure owing to either undrained shear failure or excessive settlement. Therefore, it is essential to improve soft soils by increasing their bearing capacity and reducing their compressibility. Amongst various techniques, addition of specific admixtures to the soft soil is one of the most effective and convenient methods of stabilization. This paper presents a series of experimental studies conducted for soft soil improvement using various admixtures like nylon cord fibers, banana fibers, and plastic waste materials. A comparative analysis of the enhancement of compaction and penetration properties of the treated soil was conducted. It was observed that the values of optimum moisture content, maximum dry density, and California Bearing Ratio of the soil were altered significantly by the addition of admixtures wherein China nylon cord fiber was most effective. An array of concluding remarks were derived from the overall study.

1 INTRODUCTION

Failure of foundations for infrastructure constructed upon soft and compressible soil is evidenced either by undrained shear failure of the supporting soil or excessive settlement (Chang et al., 2008). Therefore, providing a cost-effective foundation for civil infrastructure with the required factor of safety against bearing failure and an acceptable magnitude of settlement is imperative for long-term safety and serviceability (Sánchez-Garrido et al., 2022). Subsoil existing in significant regions worldwide consists of soft alluvial clay or soft marine clay deposits in coastal areas (Basack et al., 2023). Consequently, it is vital to augment the strength and stiffness of the soft soil; this technique is termed as 'ground improvement' (Basack et al., 2022a). Various types of ground improvement techniques adopted around the world have been primarily classified as mechanical stabilization, consolidation, and chemical stabilization (Basack et al., 2022b).

In the case of mechanical stabilization, soft soil is subjected to external impact energy such as compaction or blasting, blending, or replacing the poor soil with better quality materials or soil reinforcement, where stiffer materials such as compacted fine or coarse aggregates are used to strengthen the soft soil (Afrin, 2017). The consolidation technique is specifically applicable to soft clay, which includes the application of preloading on the ground surface and allowing the clay layer to consolidate. To accelerate the consolidation process, a series of vertical drains are often used to assist radial consolidation. Stone columns, sand compaction piles, and prefabricated vertical drains are included in this category of soft ground improvement techniques (Basack et al., 2022). The use of chemical additives or admixtures to enhance soft soil performance is termed chemical soil stabilization. Among the particular techniques of chemical soil stabilization are stabilization by cement, lime, and bituminous emulsion (Cabezas et al., 2019).

Using certain admixtures for soft soil has often been considered as a combination of mechanical and chemical stabilization. Apart from acting as stiffening materials to improve the overall strength and rigidity of soft dirt, admixtures often undergo chemical reactions with soil particles to alter their geotechnical properties. The application of bagasse ash, stone dust, or other similar materials used as admixtures has been a few examples of this category (Basack et al., 2021).

2 LITERATURE REVIEW

A brief overview of the concepts and field applications of soil stabilization by admixtures has been discussed by Puppala et al. (2015). The authors described a diverse array of admixture treatment techniques, encompassing the formulation of stabilizers and their respective dosages, together with experimental mix design procedures, field building practices, and quality control evaluations.

A laboratory-based investigation on the influence of fly ash and lime used as admixtures on the geotechnical properties of expansive soil was performed by Ji-ru and Xing (2002). The stabilization was visualized by alteration in soil texture, Atterberg limits, compaction parameters, and CBR. Choobbasti et al. (2010) studied the influence of rice husk ash and lime as admixtures for controlling the swelling potential of soft clay. The method was found to assist in the effective chemical reaction between the admixtures and clay particles and in the enhancement of the geotechnical qualities of soft ground.

Another technique of applying sawdust ash as admixture to increase strength of unstable soil was performed by Butt et al. (2016). This industrial waste material was observed to be a cost-effective and suitable strengthening ingredient for base and sub-base course in flexible pavement construction. Jalal et al. (2020) performed an extensive review regarding the utilization of calcium-based admixtures for expansive soil stabilization. The authors provided an in-depth analysis of the effects of calcium-based stabilizers on the physicochemical attributes of soft soils.

Renjith et al. (2021) studied the improvement of fly-ash-based soil stabilization techniques for highway building. They found that the application of enzymes as an additive improved the strength of the fly ash-treated soil. The utilization of specific industrial waste materials including steel slag, blast furnace slag, and phosphor-gypsum for treating construction residue soil was conducted by Chen et al. (2024). This research examined the engineering features of construction residual soil treated with a specific admixture, studying how organic matter concentration, dose, and curing age affect unconfined compressive strength. The long-term performance was also studied in wet–dry cycle and water stability tests.

It is an innovative approach to address plastic pollution by incorporating solid plastic waste into the subgrade pavement layers. Plastic waste can be used as a sustainable alternative in road construction, thereby reducing environmental pollution and improving waste management practices. By repurposing plastic waste in this manner, this practice contributes to circular economy principles and provides a practical solution for reducing plastic accumulation in landfills and oceans (Amena, 2022).

The application of leftover plastic materials to improve soft soils is an emerging trend. Small-sized strips or pieces derived from waste plastic bottles or other materials, used as admixtures to virgin soil, were found to provide satisfactory results in soil stabilization (Singh and Mittal, 2019; Gangwar and Tiwari, 2021). Attempts have also been made to use strips of plastic-mesh textile bags in combination with fly ash treated soil, which indicated significant increment in shear strength properties of soft clay (Bitar et al., 2024). Other recent techniques include the use of banana fiber reinforcements derived from banana trees and cut into small strips (Gobinath et al., 2020; Bawadi et al., 2020) and China nylon cord obtained from waste rubber tyres (Jafari and Esna-ashari, 2012).

3 MOTIVATION AND RESEARCH METHODOLOGY

Waste plastic materials in various forms are widely used by citizens and are available everywhere in communities (Pilapitiya and Ratnayake, 2024). A similar situation also exists for waste rubber tyres obtained from vehicles (Tian et al., 2024). In addition, banana trees exist in approximately 135 countries around the world (Li et al., 2024). Although waste plastic materials, banana trees, and waste tyres are widely available, their applications for soil stabilization are still not thoroughly investigated. An in-depth investigation of the utilization of large volumes of these waste materials as admixtures for soft soil stabilization is yet to be conducted, although the use of various conventional materials in ground improvement has been studied in detail.

This study seeks to address this research gap. This particular research has aimed to bridge this gap through a set of comprehensive laboratory tests and comparative analyses. The research methodology includes extensive experimental investigation in a geotechnical engineering laboratory with the objective of studying the effectiveness of the above-mentioned waste materials as admixtures for stabilizing soft soil. Various materials were collected in appropriate quantities, including soft soil, waste plastics, banana tree branches, and China nylon cord. Disturbed soil samples were tested to determine their engineering properties. In the next step, various admixtures were cut to the desired shapes and sizes, intimately mixed with the soil samples, and subjected to laboratory testing to observe alterations in the geotechnical parameters of the treated soils. Thereafter, in-depth analyses and interpretations of the test results were obtained, followed by a comparative study.

4 MATERIALS

4.1 SOIL

Disturbed soft soil sample in bulk volume has been collected by auger boring (ASTM, 2024) from a pit of 1 m × 1 m × 1 m dug in a suitable rural site near the city of Midnapore of West Bengal, India (Global coordinates: 22.24ºN; 87.65ºE). The soil was air-dried under sunlight for 30 days and thereafter oven-dried for 24 hours. The dry samples were finely ground and tested in the geotechnical laboratory. Particle size distribution derived from sieve analysis and hydrometer tests (ASTM, 2017) revealed 30% clay, 32% silt, and 38% sand. Figure 1 depicts the relevant curve for particle size distribution.

The geotechnical attributes of the disturbed soil sample were assessed using procedures standardized by (ASTM, 2018), as detailed in Table 1. From the unified classification system (ASTM, 2000), the soil is categorized as CL.

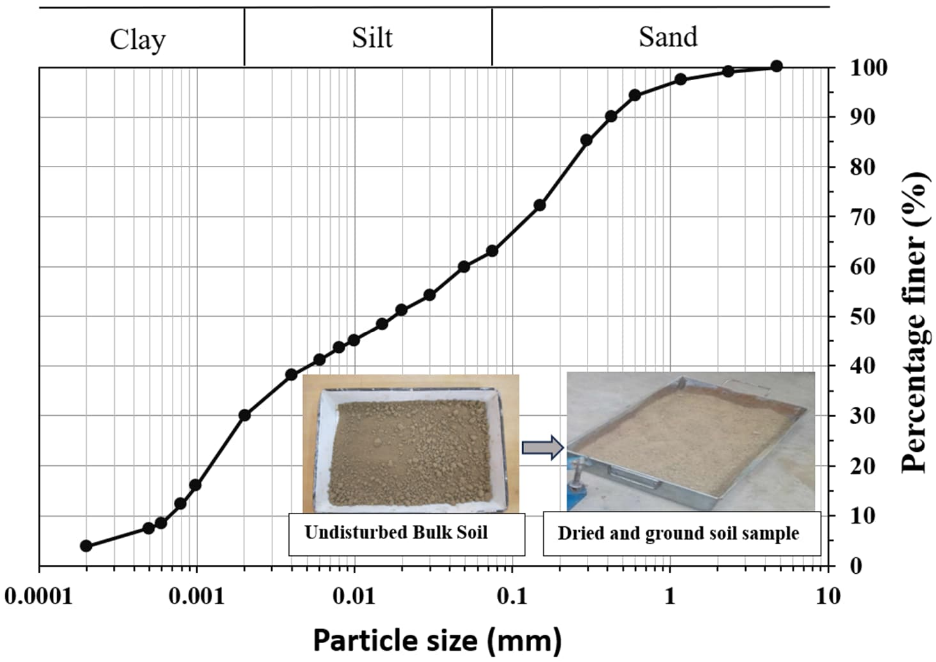


Figure 1: Particle size distribution of soil

Table 1: Geotechnical properties of soil

Parameters			**Values**
Particle size distribution	Clay		32%
	Silt		30%
	Sand		38%
Atterberg limits [a]	Liquid limit		26 ± 1%
	Plastic limit		20 ± 1%
	Plasticity index		6 ± 2%
Specific gravity of solid particles (G_s) [b]			2.5
Standard Proctor compaction test [c]	Optimum moisture content, w_o		12.1%
	Maximum dry density, $\gamma_{d_{max}}$		17.8 kN/m^3
California Bearing Ratio (CBR) [d]	Penetration [e]	2.5 mm	Un-soaked: 9.7% Soaked (4 day): 6.7%
		5.0 mm	Un-soaked: 10.2% Soaked (4 day): 7.3%
The test procedures followed [a] (ASTM 2018), [b] (ASTM 2014), [c] (ASTM 2007), and [d] (ASTM 2021). Note: [e] Considering the higher values, CBR => Unsoaked:10.2%; Soaked (4 day): 7.3%.			

The compaction curve obtained from the standard Proctor test and the load-penetration curve derived from the CBR test data are shown in Figures 2 and 3, respectively.

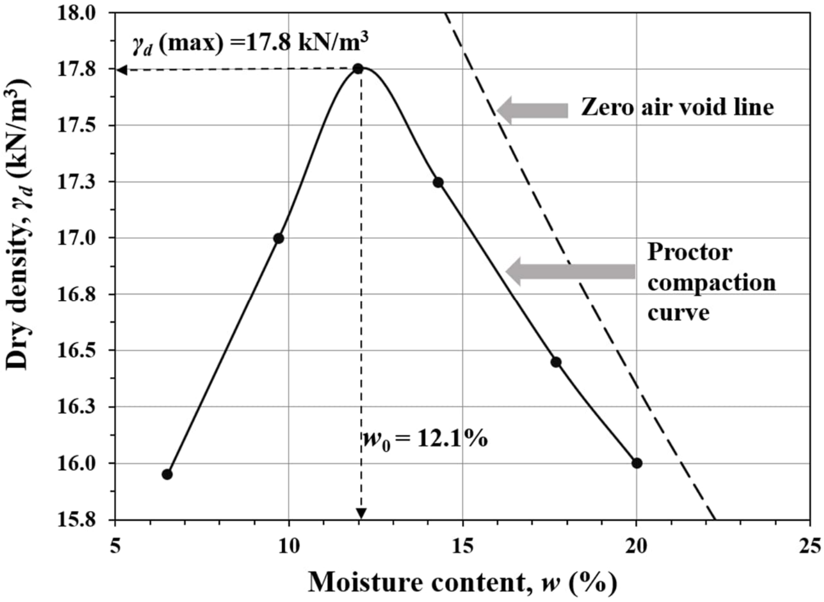


Figure 2: Proctor compaction test results

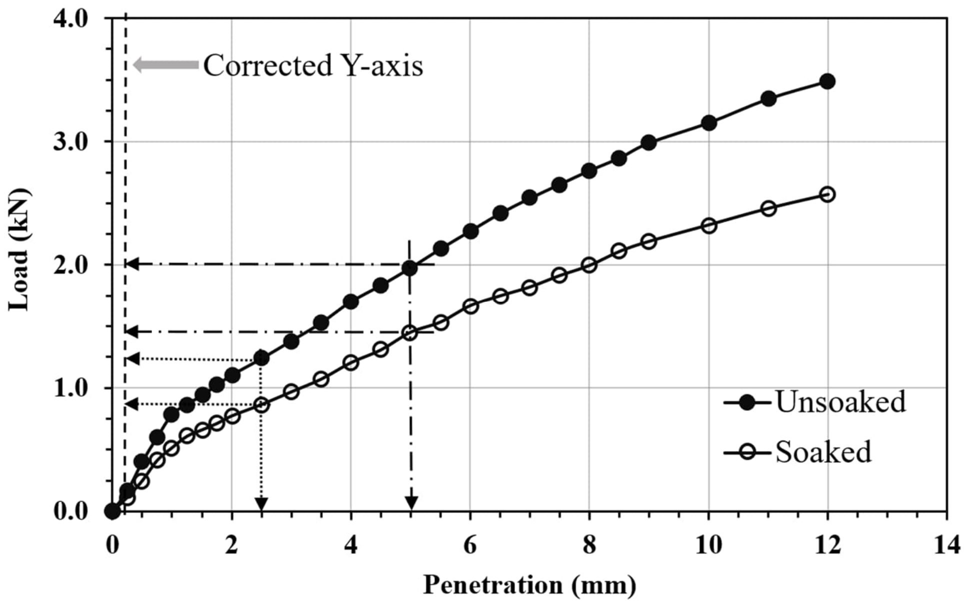


Figure 3: Load-penetration curves for CBR tests

4.2 ADMIXTURES

The admixtures used were derived from: (i) waste plastic bottles, cut to desired shapes and sizes or grounded in powder form, (ii) fibers derived from China nylon cord obtained from waste rubber tyres of vehicles, and (iii) fibers obtained from branches of banana trees, cut to desired sizes. Photographic views of the admixtures are portrayed in Figure 4.

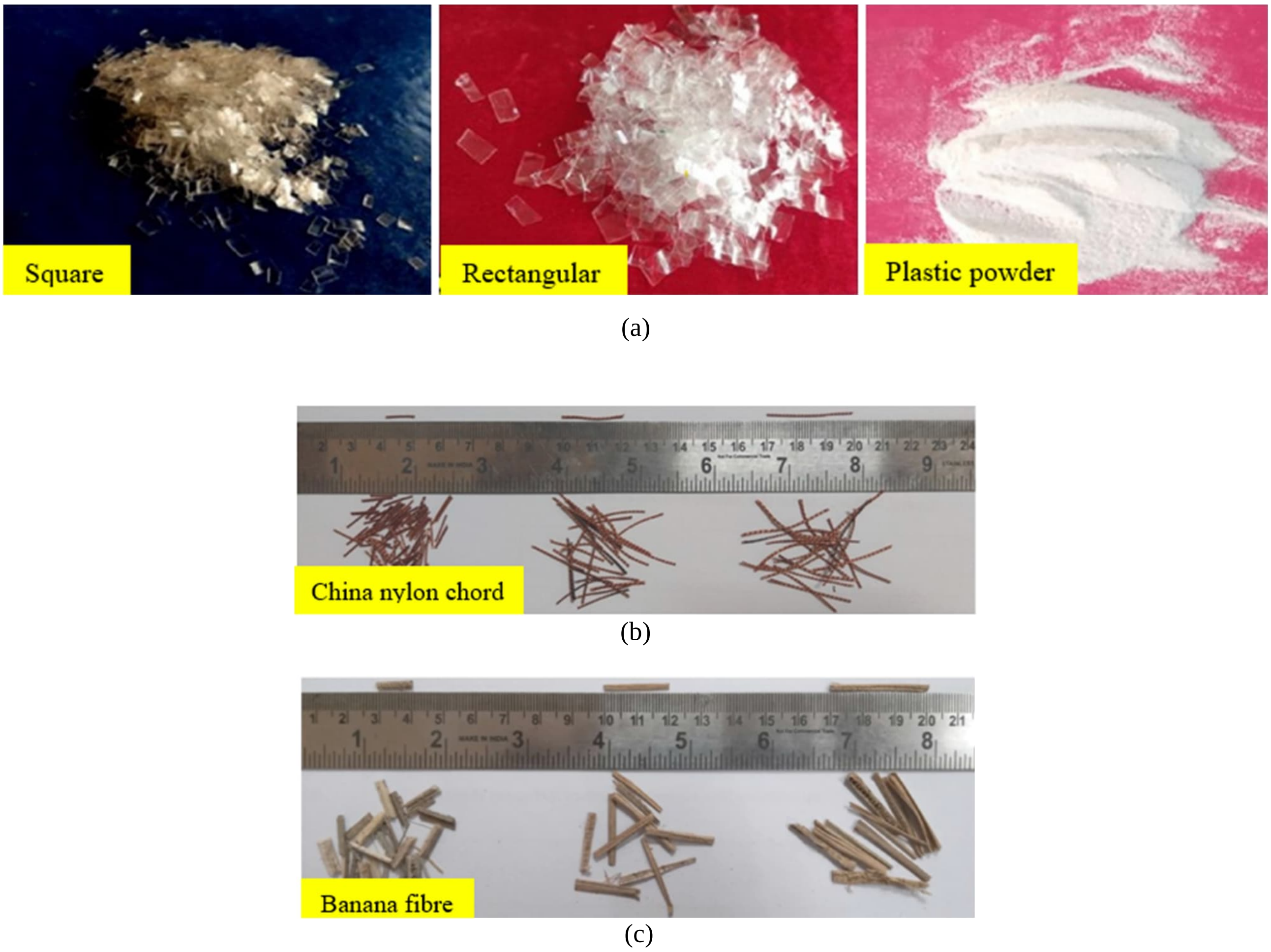


Figure 4: Photographic views of different admixtures: (a) Plastic materials, (b) China nylon cord, and (c) Banana fibers

The specific gravity of each admixture was ascertained using the pycnometric method (ASTM, 2014). In addition, plastic, nylon cord, and banana fibers were cut into wires of specified sizes and tested in the laboratory to determine the Young's modulus following a standard procedure (ASTM, 2020). The characteristics of these substances are presented in Table 2.

Table 2: Engineering properties of admixtures

Properties	Materials	Values
Specific gravity	Plastic waste	1.42
	China nylon cord	1.38
	Banana fiber	1.35
Young's modulus	Plastic waste (square/rectangular)	2.92 GPa
	China nylon cord	0.06 GPa
	Banana fiber	19.4 GPa

4.2.1 Plastic Wastes

The admixtures derived from waste plastic bottles are used in three different specifications:

- Square, having size 5 mm × 5 mm
- Rectangular, having size 10 mm × 5 mm
- Finely ground to powder using a high-speed blender, with the particle sizes ranging from 75 μm to 1.18 mm

4.2.2 China Nylon Cord

The synthetic nylon cord obtained from waste vehicle tyres was cut to various sizes having diameter of 1.0 mm and lengths of 10, 20, and 30 mm. Attempts have also been made to use finely ground powder derived by blending cords, but the same was unsuccessful. Owing to its soft and brittle nature, blending in bulk volume was difficult, which affected the workability. A similar problem was found in a few previous studies (Wealthy Waste, 2024).

4.2.3 Banana Fiber

The banana fibers are derived from the stems of banana trees in bulk quantities, each fiber having a diameter of 1.5 mm and lengths of 10, 20, and 30 mm. Similar to the nylon cord, as explained above, the fibers were not used in powdered form.

5 EXPERIMENTAL PROGRAM, RESULTS AND DISCUSSION

The primary aim of the experimentation was to determine the compaction and penetration characteristics of the soil treated with the different admixtures described above at specified quantities. The details of the test arrangement are listed in Table 3. The outcomes derived from laboratory experimentations are given in Table 4 and their analyses and interpretations are presented as well.

Table 3: Test program

Particulars				Tests [b]
Admixture	Shape	Size	Mix proportion (%) [a]	
Untreated soil	-	-	-	P, C_{US}, C_S
Plastic waste materials	Square	-	1, 2, 3, 4	P, C_{US}, C_S
	Rectangle	-	1, 2, 3, 4	P, C_{US}, C_S
	Powder	-	1, 2, 3, 4	P, C_{US}, C_S
China nylon cord	-	Length: 10mm, 20 mm, 30 mm	0.5, 1, 1.5, 2	P, C_{US}, C_S
Banana fiber	-	Length: 10mm, 20 mm, 30 mm	0.5, 1, 1.5, 2	P, C_{US}, C_S

Notes: (1) Total number of test sets: 37. (2) For each test set, 3 individual tests were conducted to minimize the error. (3) [a] Quantities are measured by weight of soil. (4) [b] P: Standard Proctor compaction test; C_{US}: CBR test un-soaked; C_S: CBR test soaked.

Table 4: Standard Proctor Compaction and California Bearing Ratio test results

Admixture	Shape	Size	Mix Proportion (%)	w_0 (%)	$\gamma_{d_{max}}$ (kN/m^3)	Unsoaked CBR (%)	4 Day Soaked CBR (%)
None	-	-	-	12.1	17.8	10.2	7.3
Plastic wastes	Square	-	1	11.9	17.3	10.9	8.0
		-	2	11.3	16.9	11.8	8.6
		-	3	10.8	16.5	11.3	8.5
		-	4	10.3	15.9	10.9	8.4
	Rectangular	-	1	11.5	17.5	10.9	8.1
		-	2	10.9	17.1	11.4	8.4
		-	3	10.3	16.9	11.2	8.3
		-	4	9.6	16.3	11.1	8.1
	Powdered	-	1	11.8	17.1	11.6	8.9
		-	2	11.1	16.7	12.4	9.0
		-	3	10.6	16.3	12.2	8.8
		-	4	9.9	15.6	11.8	8.6

China Nylon Cord	-	Length = 10 mm	0.5	11.6	17.2	12.0	9.3
			1.0	11.5	16.8	12.9	9.9
			1.5	11.2	15.9	12.6	9.7
			2.0	10.6	15.5	12.1	9.5
		Length = 20 mm	0.5	11.8	17.3	12.2	10.2
			1.0	11.7	17.1	13.2	11.0
			1.5	11.3	16.5	13.0	9.9
			2.0	10.8	16.0	12.4	9.6
		Length = 30 mm	0.5	11.9	17.3	12.2	10.0
			1.0	11.8	17.2	12.7	10.2
			1.5	11.6	16.6	12.4	9.9
			2.0	11.1	16.2	12.3	9.8
Banana Fiber	-	Length = 10 mm	0.5	12.9	17.3	11.4	9.1
			1.0	14.0	16.3	12.2	9.7
			1.5	15.5	15.7	11.9	9.1
			2.0	16.8	15.2	11.4	8.9
		Length = 20 mm	0.5	13.4	16.5	11.8	9.0
			1.0	14.9	16.2	12.4	10.2
			1.5	16.5	15.3	12.2	9.4
			2.0	17.3	14.8	11.9	9.0
		Length = 30 mm	0.5	14.7	16.2	11.7	8.8
			1.0	15.7	15.8	12.1	10.0
			1.5	17.2	15.0	12.0	9.5
			2.0	18.4	14.6	11.6	9.0

5.1 COMPACTION

Figure 5 depicts a few representative proctor curves obtained from standard Proctor compaction tests (ASTM, 2007). The trends of Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) of the treated soils with different admixtures in various proportions and lengths are illustrated in Figures 6, 7 & 8.

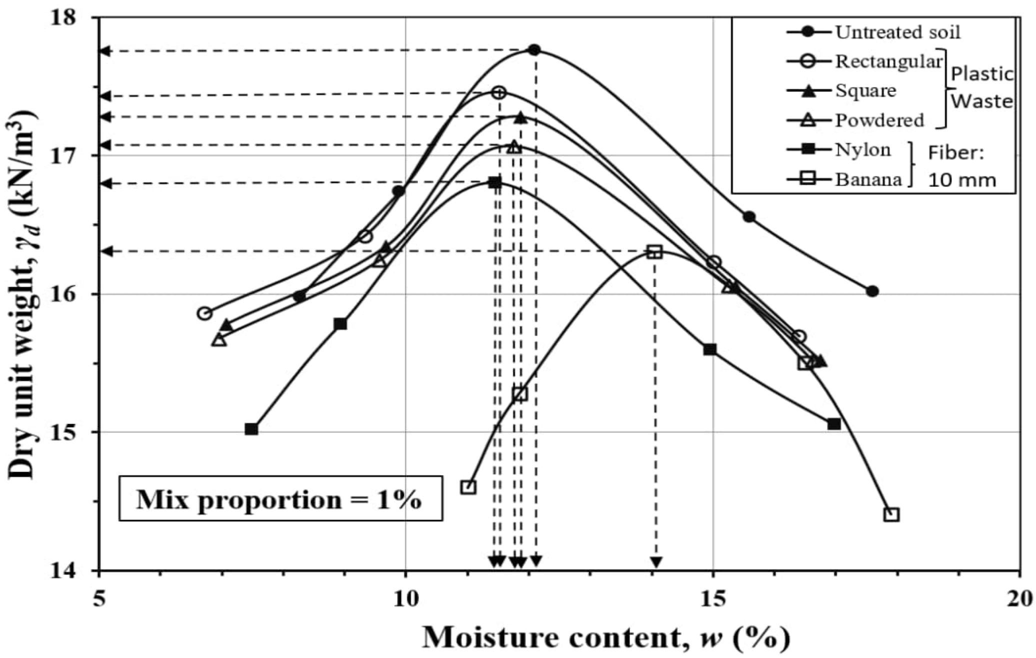


Figure 5: Standard Proctor compaction curves for representative samples

The variation in the compaction properties of treated soil has been studied by a group of non-dimensional parameters (Basack et al., 2021) given as follows:

$$\alpha_o = \frac{(w_o)_t}{(w_o)_u} \quad (1)$$

$$\alpha_d = \frac{(\gamma_{d_{max}})_t}{(\gamma_{d_{max}})_u} \quad (2)$$

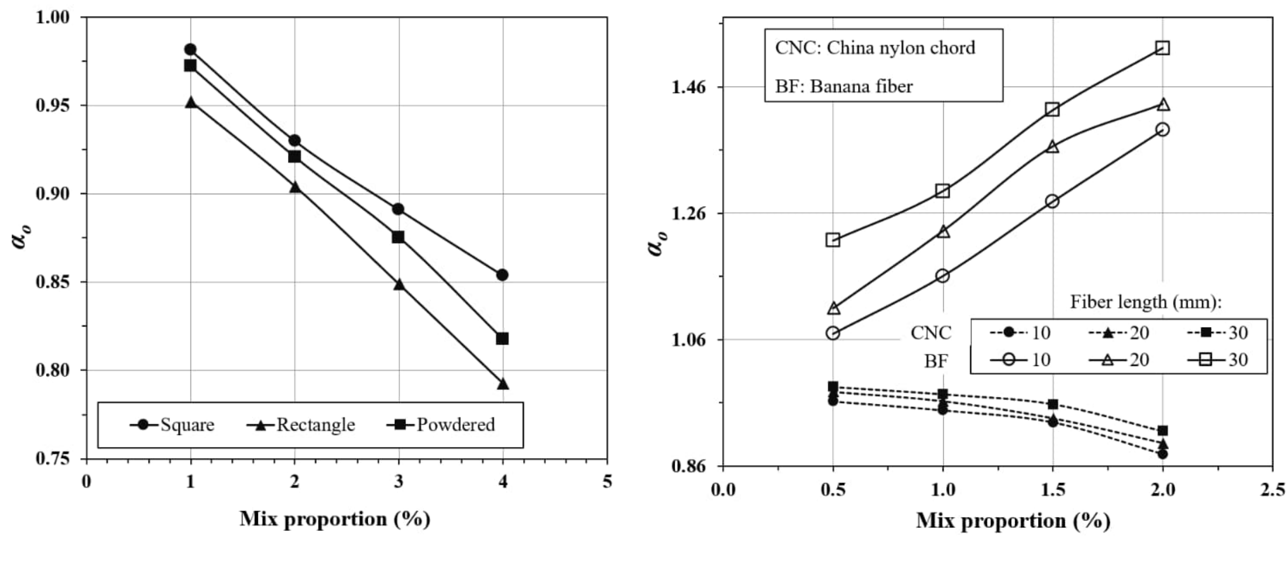
where, α_o and α_d are the normalized values of optimum moisture content and maximum dry density, w_0 and $\gamma_{d_{max}}$ are the optimum moisture content and the maximum dry density and the suffixes *t* and *u* referred to the values relevant to treated and untreated soils, respectively.

The variation of α_o and α_d with mix proportion for plastic waste, China nylon cord, and banana fiber are portrayed in Figures 6 and 7, respectively.

The ranges of variation of the values of α_o and α_d were $0.79 \le \alpha_o \le 0.98$, $0.88 \le \alpha_o \le 0.99$, $1.07 \le \alpha_o \le 1.52$, $0.88 \le \alpha_d \le 0.98$, $0.87 \le \alpha_d \le 0.98$, and $0.82 \le \alpha_d \le 0.97$ for plastic waste, nylon, and banana fibers, respectively. As observed from Figures 6 and 7, the values of both parameters α_o and α_d decreased with increasing mix proportion for plastic waste and nylon. In the case of banana fiber, the parameters α_o and α_d were observed to increase and decrease, respectively, with the mix proportion.

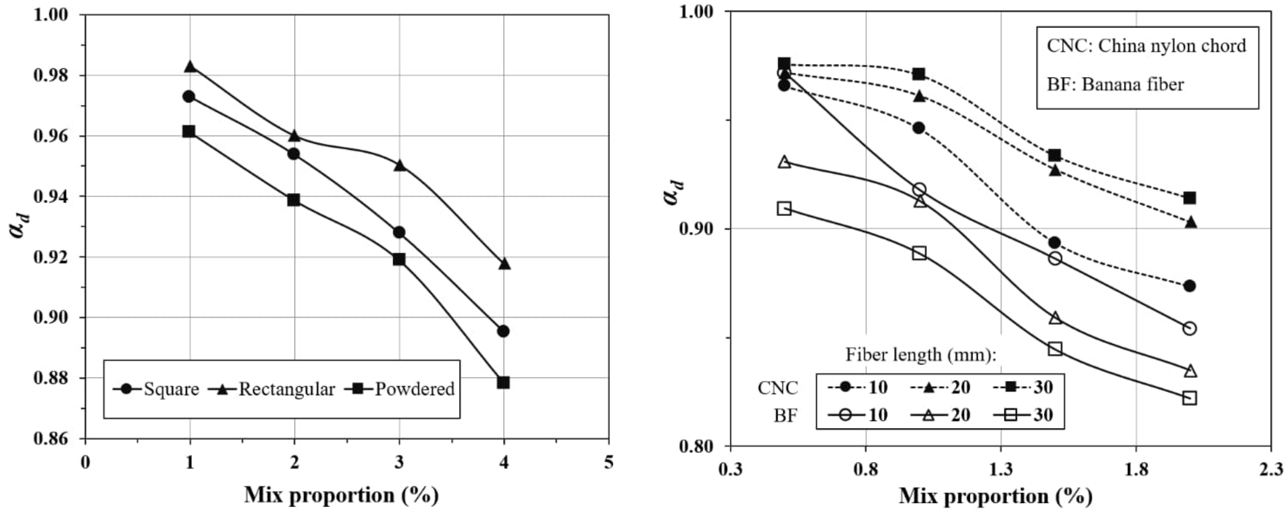
Both plastic wastes and nylon are chemically inert with soil mass, with negligible water retention capacity. In addition, their specific gravity is less than that of soil particles. These factors may have significantly reduced OMC and MDD of the treated soil. The incorporation of banana fibers into the soil can influence its physical properties owing to their high water retention capacity. Unlike plastics and nylon, the water retention capability of banana fibers can increase the OMC because they hold more moisture within the soil matrix. This addition tends to decrease the MDD as it changes the overall density distribution of the soil (Maqbool et al., 2023; Patil and Pusadkar, 2020).

The variation of α_o and α_d with the lengths of the nylon and banana fibers is depicted in Figure 8. Both parameters α_o and α_d were found to increase with the length of the nylon fiber. The observed pattern can be justified by the fact that the ascending length of the fiber possibly initiated a rearrangement and realignment of soil elements, thus enhancing the OMC and MDD of the treated soil (Owino and Hossain, 2023). In addition, nylon fibers being chemically inert with soil mass produced insignificant variations in the values of α_o and α_d (Zafar et al., 2023). Conversely, for Banana fiber, the parameters α_o and α_d were observed to increase and decrease with length, respectively. However, no specific pattern of variation could be seen. The rate of increment was observed to be sharper for banana fiber compared to nylon.



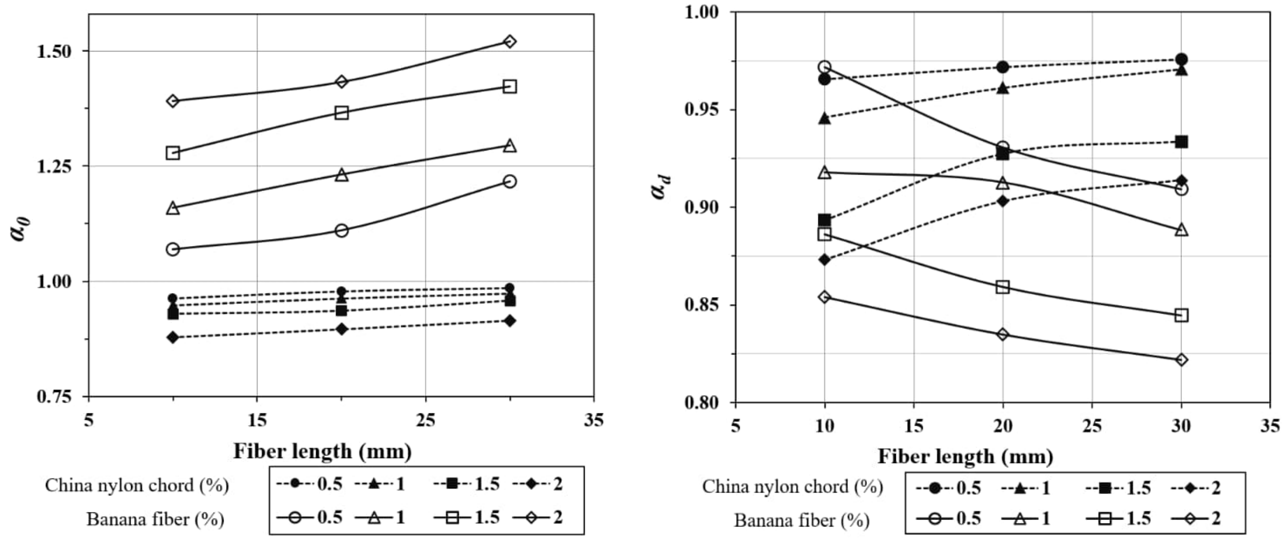
(a) (b)

Figure 6: Variation of α_0 with mix proportion in case of: (a) plastic waste, and (b) China nylon cord and banana fiber.



(a) (b)

Figure 7: Variation of α_d with mix proportion in case of: (a) plastic waste, and (b) China nylon cord and banana fiber



(a) (b)

Figure 8: Variation of normalized parameters: (a) α_0 with fiber length (b) α_d with fiber length

5.2 PENETRATION

The penetration characteristics of the stabilized soil were characterized through CBR tests wherein both the unsoaked and soaked (4 day) soils were tested in accordance with ASTM, 2021. The variation in CBR strength for the various soil-admixture combinations is graphically represented in Figures 9 and 10.

For comparison, the CBR values were normalized with respect to the untreated soil (Basack et al., 2021) given as follows:

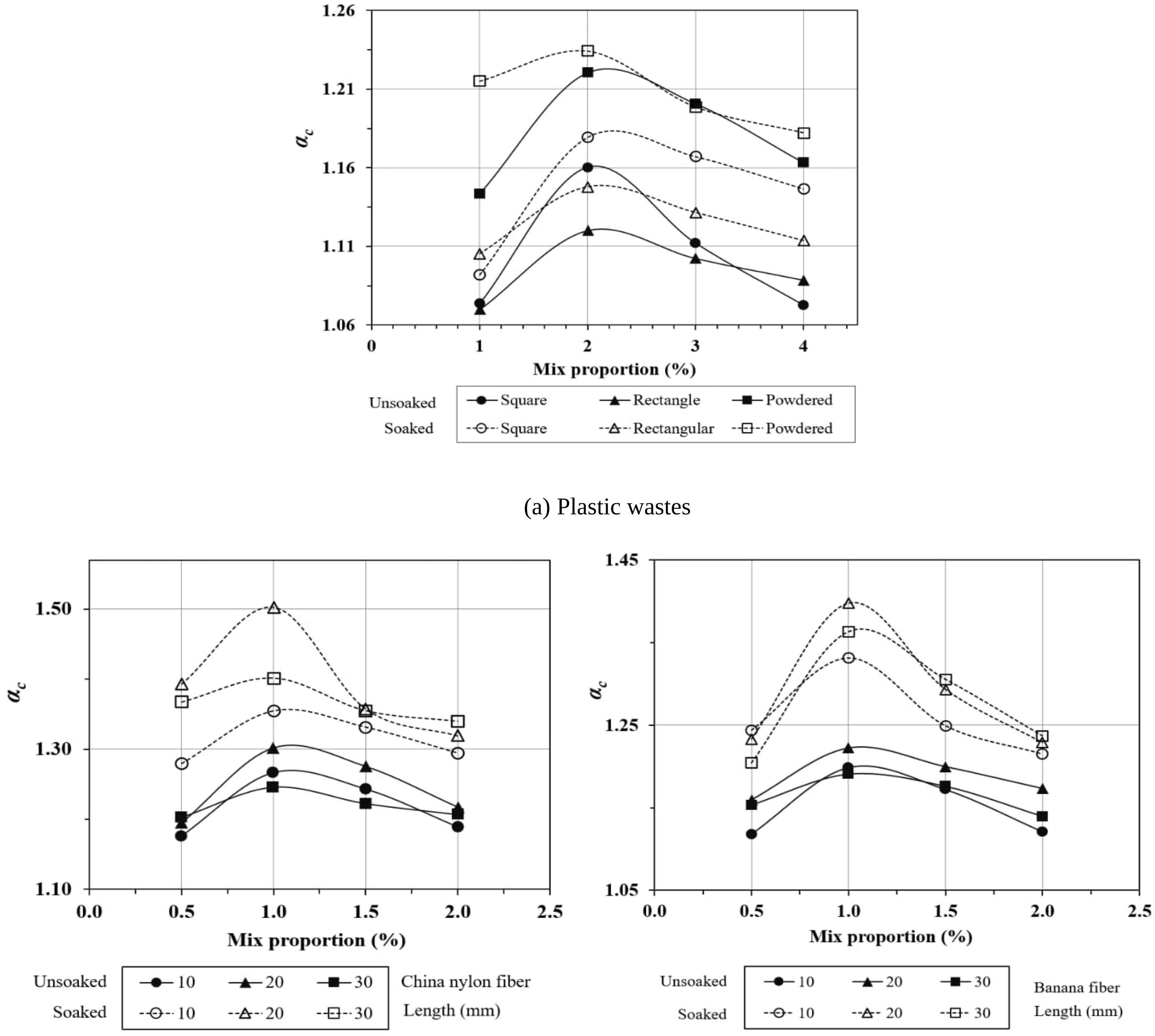
$$\alpha_c = \frac{CBR_t}{CBR_u} \tag{3}$$

where, α_c is the normalized CBR, and CBR_t and CBR_u are the values of CBR for treated and untreated soils, respectively.

The variation of α_c with mix proportion for different admixtures have been plotted in Figure 9. As observed from the plot, the patterns of variation have been curvilinear. With the chosen mix proportion, the normalized values of un-soaked and soaked CBR were found to vary between the ranges of $1.07 \leq \alpha_c \leq 1.22$ and $1.09 \leq \alpha_c \leq 1.23$ for plastic wastes, while $1.18 \leq \alpha_c \leq 1.30$, $1.28 \leq \alpha_c \leq 1.50$, $1.12 \leq \alpha_c \leq 1.22$, and $1.20 \leq \alpha_c \leq 1.40$ in the cases of nylon and banana fibers, respectively. Also, with ascending mix proportion of the additives, the parameter α_c initially increased, attained peak values and thereafter decreased, for all the additives. The peak values were observed to occur in mix proportions of 2% - 2.2% and 1% - 1.2% for plastic wastes and fiber admixtures, respectively.

Based on the observed variation in α_c with the fiber length (Figure 10), the peak values of α_c were attained for a fiber length of 20 mm for both fiber types.

The penetration behavior of the stabilized soil was evaluated by CBR testing. Keep in mind that the CBR test is well known for its intrinsic variability and limited repeatability, as demonstrated by the research background for the DCP-DN design method for LVSR (Paige-Green and Zyl 2019). Given the documented exposure levels on test repeatability, CBR values in this study have been rounded to a single decimal place so that the analysis produces a realistic order of engineering accuracy and does not over-fit the experimental data.

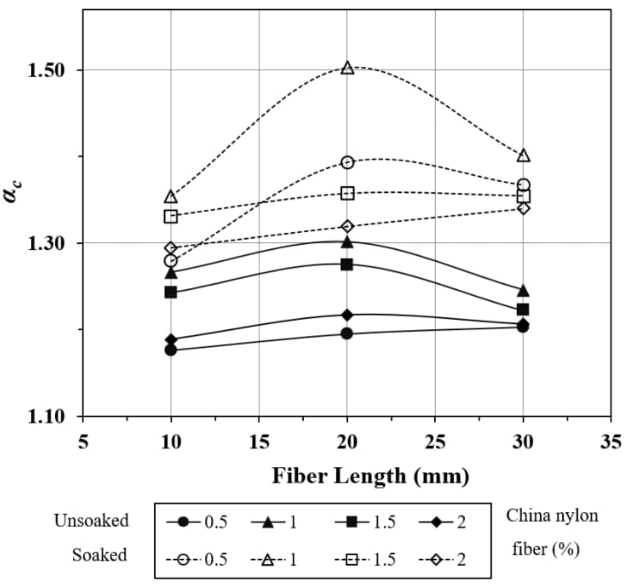


(a) Plastic wastes

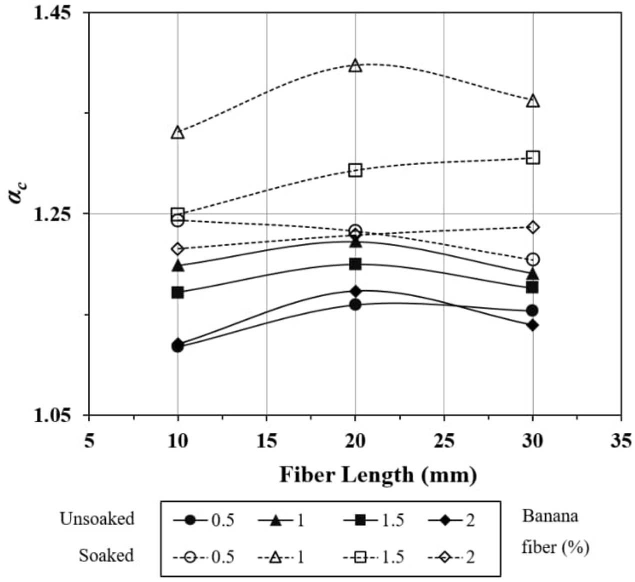
(b) China nylon Fiber

(c) Banana Fiber

Figure 9: Variation of α_c with mix proportion



(a) China nylon Fiber



(b) Banana Fiber

Figure 10: Variation of α_c with fiber length

The above observations may be explained by the possibility that an increase in admixture proportion initially enhances the soil stiffness due to the reinforcing effect until the peak value is attained, whereas a further increase in additive content subsequently reduces the resistance to penetration owing to the excessive admixture content (Kassa et al., 2020). Similar observations were also reported by several previous studies (Badiger et al., 2019; Nezhad et al., 2021; Ahmed et al., 2024).

Using plastic waste as admixtures enhances the CBR of unstable soil primarily due to tensile reinforcement of discrete plastic admixtures randomly distributed in the soil mass, initiating reduced swelling, enhanced frictional resistance and filling voids. The soaked CBR tests simulate the worst-case scenario for subgrade, when the soil is fully saturated, for example during monsoon or high water table conditions. In the current study, it is observed that the required admixture contents and sizes for optimal values of soaked CBR are more than those for unsoaked tests, possibly due to counteracting the weakening effects of water saturation (Gangadhara and Vivek 2016; Boobalan et al. 2023).

6 ANALYSIS AND INTERPRETATION

The above experimental findings reveal that both the plastic wastes and nylon fibers, being chemically inert with soil mass, OMC and MDD descended with ascending admixture content. This implies a reduced volume of water required for field compaction and a reduction in the effective overburden pressures in the deeper soil layers. Conversely, the variance pattern differed for banana fiber due to an increase in OMC and a decrease in MDD. Similar observations were found in few recent studies as well (Çelik et al., 2025; Akıllı et al., 2025; Özdemir et al., 2024).

Furthermore, chemically inactive admixtures eliminate the possibility of biodegradation and subsequent alteration in the treated soil properties. Besides the direct mechanical effects of using solid waste materials, their influence on the environmental background for soil should also be taken into account. Although the chemical inertness of nylon and plastic waste would resist a rapid biodegradation which is desired to maintain the enhanced geotechnical properties, their long-term physical existence presents concerns regarding microplastic accumulation. During long engineering lifecycles, it is conceivable that environmental stress factors might result in the fiber breaking up. But in the case of ground improvement, these materials are intimately mixed and encapsulated in a compacted soil. This burial effectively protects the admixtures from ultra-viiolet radiation and significant mechanical abrasion, the two principal drivers of plastic degradation in surface environments (Vincenzini et al. 2021; Amena 2022; Kalita et al. 2025; 2026). As a result, the potential of microplastic leaching toward the surrounding environment reduces with time, thus indicating that these waste-derived admixtures might serve as long-lasting and environmentally-friendly solution for stabilizing soft soils. Thus, considering the enormous efforts in cutting the bulk volume of square- or rectangular-shaped plastic waste, as well as powdered plastic waste, nylon fibers appear to be a preferable option. Moreover, from the CBR test results, the mix proportion relevant to the peak value is preferable.

7 CONCLUSIONS

A comparative experimental study was performed to investigate the effectiveness of selected types of admixtures applied to improve the compaction and penetration characteristics of soft clayey soil. Plastic waste materials derived from waste bottles and cut into the shapes of square, rectangular and powdered form, China nylon fibers derived from waste rubber tyres, and banana fibers cut from banana stems, were intimately mixed with saturated soft clay to stabilize the soft soil.

The main focus of the current work is to conduct an in-depth comparative study on the influence of plastic wastes, natural and synthetic fibers on soil stabilization in terms of compaction and penetration characteristics. The specific effects of admixture content and their sizes on the compaction parameters (OMC and MDD) and CBR (unsoaked and soaked) were analysed through non-dimensionalized parameters α_o, α_d and α_c and the relevant conclusions are drawn.

Standard Proctor compaction and California Bearing Ratio (CBR) tests were conducted on both untreated and treated soil samples. This study revealed that both the compaction and penetration characteristics of the soil were significantly influenced by the addition of admixtures. The values of both optimum moisture content (OMC) and maximum dry density (MDD) decreased with the increase of waste plastic and nylon contents. The values of both parameters α_o and α_d descended with ascending mix proportion for plastic waste and nylon. In the case of banana fiber, the parameters α_o and α_d were observed to increase and decrease, respectively, with the mix proportion. Both parameters α_o and α_d were found to increase with the length of the nylon fiber. Both plastic waste and nylon are chemically inert relative to soil mass and possess minimal water retention capability. Their specific gravities are lower than those of the soil particles. These effects likely contributed to the reduction of OMC and MDD of the treated soil to a considerable extent.

However, banana fiber exhibits superior water retention capabilities, and its specific gravity is less than that of soil particles. These effects likely increased OMC and decreased MDD of the treated soil significantly. For banana fiber, the parameters α_o and α_d increased and decreased, respectively, with the mix proportion. In contrast, for banana fiber, the metrics α_o and α_d were noted to increase and decrease with length, respectively. No clear pattern of variation could be identified. The rate of increment was observed to be more pronounced for banana fiber, compared with nylon.

As far as the CBR tests are concerned, the values of both the unsoaked and 4 day soaked CBR varied nonlinearly with the admixture quantity and fiber length. With increasing quantity of admixture, the CBR initially increased, attained peak values, and thereafter decreased. The peak values occurred at mix proportions of 2.0%–2.2% and 1.0%–1.2% for plastic wastes and fiber admixtures, respectively. In the case of the fibers, the peak value of CBR was attained for a fiber length of 20 mm.

Overall, this investigation demonstrated that China nylon cord acted as the most effective admixture, optimizing the compaction and penetration characteristics of soft soil.

For a more comprehensive study, it is desired to carry out field based investigation and a comparative cost analysis, so as to develop appropriate design recommendations with appropriate charts and curves.

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CRediT authorship contribution statement

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STRUCTURAL CONTROLS OF THE OTWAY RANGES AND HAZARDS FOR ROAD USERS

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ABSTRACT

The Otway Ranges have long been established as a region with significant landslide hazards to both private property and public asset owners. Owing to the folded nature of the interbedded sedimentary Cretaceous Eumeralla Formation, dip-slope behaviour governs a large percentage of the slopes, and this is generally well documented by others. Along the southeast facing coastline of the ranges a significant proportion of the Great Ocean Road has been constructed sub-parallel to the strike of bedding and it is unsurprising that planar sliding on bedding is a common slope control that informs hazard assessments.

For inland routes and sections of roadway orientated outside of the kinematic window for dip-slope failure, structural controls on cut and buried slopes are often present. These are typically associated with shear and fault zones and in some cases the interaction with weaker siltstone and mudstone beds. The structural controls of two case studies are presented, each highlighting a unique set of hazards to road users. Each case study highlights the importance of establishing the structural trends, even when assessing slopes formed with fill and buried landforms. These trends then inform the engineering geological model and ultimately the assessment of hazards to road users.

1 INTRODUCTION

The Otway ranges in Victoria, Australia, are home to the Great Ocean Road (GOR), which adds significant value to the tourism industry. The State Government has active strategies in place to improve safety in the Otway Ranges (Great Ocean Road region strategy (planning.vic.gov.au) 2024) and was actively involved in the Wye River and Separation Creek rebuild following the Christmas 2015 bushfires. The Department of Transport and Planning (DTP) is tasked with management of roadside geotechnical hazards for the GOR and key inland routes.

The structural trends along the GOR have been well documented by Medwell (1968), Cooney (1982) and Edwards et al (1996). The works reported by the preceding authors are broadly focussed on major folds, faults and bedding trends with less detail on fault and shear systems, Figure 1. Furthermore, large planar failures on dip slopes are well established as a mode of failure, especially along the GOR where bedding is often undercut by wave driven erosion and the construction of the GOR itself (Williams & Muir, 1972). The extensive mapping of bedding trends is helpful as typical defect sets are consistent with literature in folded sedimentary rocks (Fookes, et al 2000). The present author's approach to hazard mapping in the Otway Ranges places a priority on locating intact rock in the study area and establishing bedding trends as a minimum to begin understanding structural controls at each site.

The two case studies presented are located within deposits of the Cretaceous Eumerella Formation (Edwards, 1996). The formation is composed mainly of fine to medium grained sandstone and siltstone interbedded with thinner and less frequent mudstone. The quartz content is relatively low and the deposits weather rapidly to sands and clays. The siltstones are notably of lower strength, slake prone and more readily erodible in comparison to the sandstone (Gill, 1979).

Edwards (1996) outlines the broad composition and physiography of the Otway Ranges as follows:

- The *"ranges are composed of uplifted and eroded Cretaceous Eumeralla Formation"*
- Miocene compression activity has produced northeast trending anticlinoria
- The southeastern limb of these folds often forms dip slopes in proximity to the coastline
- Numerous folds are offset by faults. Typically streams run sub-parallel to these fault systems.

The two cases studies aim to highlight the structural controls of slopes with aspects orthogonal to bedding. The first case study highlights that structure may control landslide geometry but not the mode of failure. It includes a translational failure in a fill batter built across a gully. The gully is controlled by a buried fault and the identification of the structure assisted with limiting the zone of remediation. The importance of historical publications that identified last interglacial sea levels and how this can affect slope remediation is discussed. The second case study was a thin wedge failure in weathered rocks in proximity to major regional folds and faults, where the structure directly controlled the landslide.

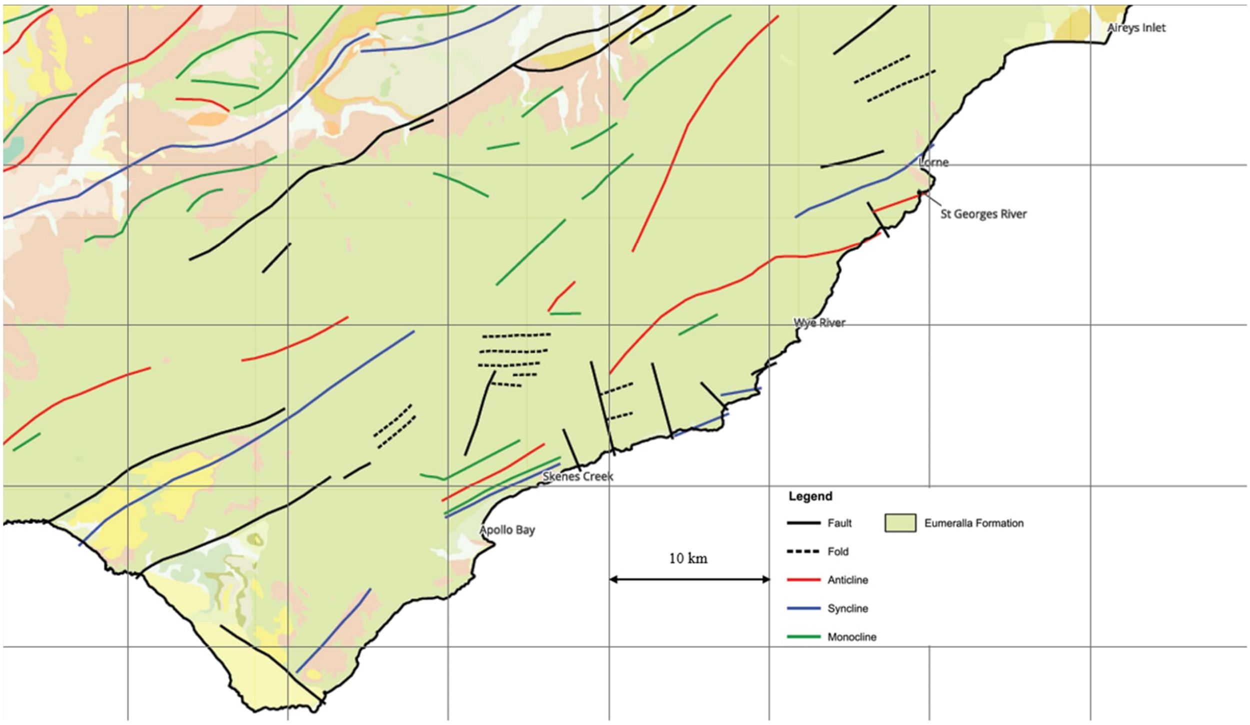


Figure 1: Otway Ranges with published major structure from Edwards (1996) and Medwell (1968). Surface Geology from Welch, et al (2011).

The hazard mapping was completed using the VicRoads Roadside Geotechnical Hazard Risk Management Guideline (VicRoads, 2018).

2 CASE STUDY 1: WYE RIVER

The GOR is a key route between Lorne and Apollo Bay and provides both community and industrial access with a significant portion of heavy rigid vehicles using the road. This section of the GOR is considered particularly important for emergency egress during bushfires and other emergency events.

The fill embankment at CH 62.54 km extends for approximately 50 m over a gully that has formed within a small amphitheatre along the coast. The site was inspected by DTP in June 2021 and pavement cracking and a slump in the fill was documented, Figure 2 (a). At that time, the DTP risk assessment identified hazards associated with vehicles impacting a stepped surface of at least 200 mm in the Apollo Bay-bound lane and single lane to road closure associated with the fill slope failure regressing into the Apollo Bay-bound lane. The assessed risk levels were in the range of “moderate to high”. A geotechnical investigation scoped by DTP targeted the landslide in the fill and the relationship between the pavement cracking and the landslide.

Cut slopes on the western side of the road exposed weathered sandstones to the gully location, Figure 2 (b) and Figure 3. To the east of the embankment is a sub-horizontal coastal shore platform and gently inclined sand-covered beach, Figure 2 (b) and Figure 3. At low tides and with sand removed from the beach, an interbedded sequence of siltstone/sandstone extends to the north of the site.

2.1 SITE INVESTIGATION

During the 2021 DTP site walkover significant cracking of the pavement was identified (refer to the red cracking on Figure 2 (a) and Figure 3) as well as relatively narrow landslides at the crest of the fill batter, Figure 2 (b) and the yellow shaded areas on Figure 3. These landslides commenced immediately north of the inferred location of the fault, Figure 3. The back scarps of the landslides were steep (approximately 63°) and the translational slides mapped indicated evidence that movement had occurred in a “two wedge” mechanism, Figure 5.

DTP completed a preliminary site investigation of four boreholes to depths of up to 12.6m below the pavement level. The Author completed a walkover to conduct geomorphological mapping and improve the understanding of local structural trends. Importantly, in 2019, the Author had mapped the area for a Morley Avenue landslide assessment and at a time when there was very little sand on the beach.

During development of the conceptual geological model the abrupt transition in depth to bedrock was evident in the boreholes, Figures 4 and 5. This occurred broadly where the Wye River bound lane turns towards the north off the Point Sturt headland and immediately adjacent to a steep gully to the west of the roadway, Figure 3. The boreholes on either side of the blue fault zone (dip/dip direction of 70 to 80/020 to 030), Figure 3, indicated that the depth to bedrock changes from 2.0 m at BH96 to 6.7 m at BH97 over 12 m horizontally. On review of the LiDAR it was evident that a gully with moderate to steep convergent slopes was immediately adjacent to the fill slope. At this time, the depth to bedrock in areas of observed pavement steps/cracks, Figure 3, was not understood. A secondary targeted investigation was completed in this area and which identified that bedrock was very shallow in the Wye River- bound lane (0.5m below pavement) and up to 2 m deep in the Apollo Bay-bound lane, Figure 4.

Figure 2: (a) Pavement cracking in proximity to fill failure looking towards Morley Avenue (b) Looking south towards the Wye River headland with undercut fill slope highlighted

2.2 GEOLOGICAL MODELS

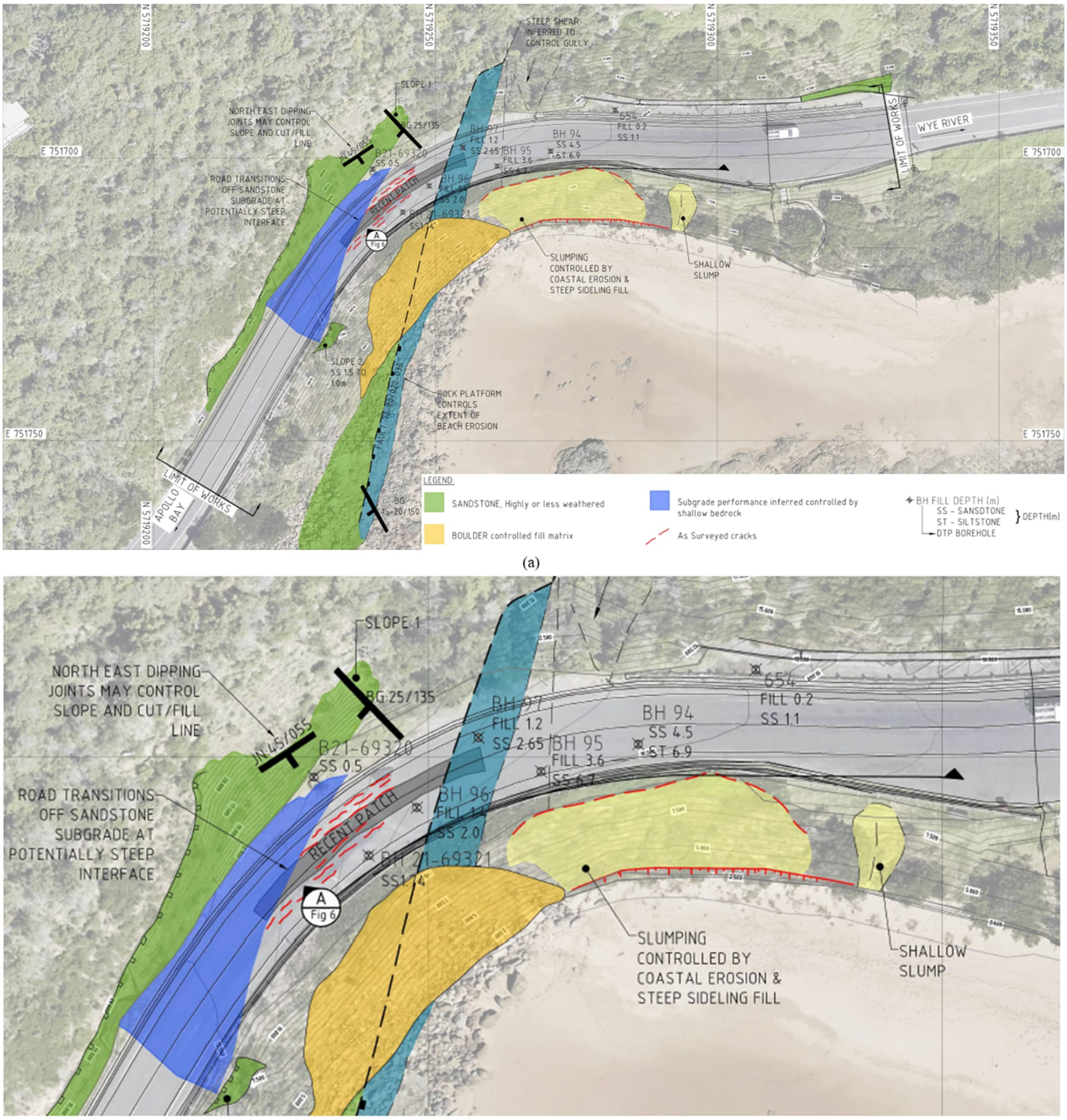
A site-wide engineering geological model combining interpretation of LiDAR survey, field mapping and the results of the intrusive investigation allowed for a thorough assessment of potential mechanisms to be undertaken. The geotechnical units identified in cut slopes and boreholes in the assessment area included:

- Fill associated with the GOR construction. The Fill was observed to be boulder dominant clast supported in zones where cuts had been made in Sandstone and matrix supported and clay dominant soils where a gully was inferred to have been backfilled, Figure 3
- Residual/Colluvial: predominantly a Sandy Clay of low to medium plasticity
- Sandstone: fine to medium, orange, brown and grey, low to medium strength, highly to moderately weathered. Blocky rock mass with conchoidal "onion skin" weathering product about the surface exposures
- Siltstone: Logged as a hard clay of low plasticity (extremely weathered) in boreholes. Where visible on the rock platform the beds had low strength and were moderately weathered.

Two cross sections that summarise the model at conceptual remediation stage are included in Figures 4 and 5.

The thickness of soil cover notably increased immediately adjacent to a steep gully and where there was an abrupt change in both the headland and sideling fill slope aspects, Figure 3. A long section at the inferred location of the buried gully and used in design is presented in Figure 6. Note that the "Design" surface was developed using Maptek GeologyCore and is a triangulation of observation points in boreholes and the shore platform.

DTP selected a pre-emptive treatment of a rock socketed cantilevered bored pile retaining wall and capping beam and with upgrades in surface and sub-surface drainage. The "post construction" surface, Figure 6, represents the profile developed from borehole, pile and shore platform records following the retaining wall remediation. Although the transition was expected in design, pile records indicated the depth to bedrock changed on a much steeper gradient most likely associated with the dip/dip direction of the fault zone.



(a)

(b)

Figure 3: (a) Site plan with pavement cracking in red and translational slides in yellow (b) Detail

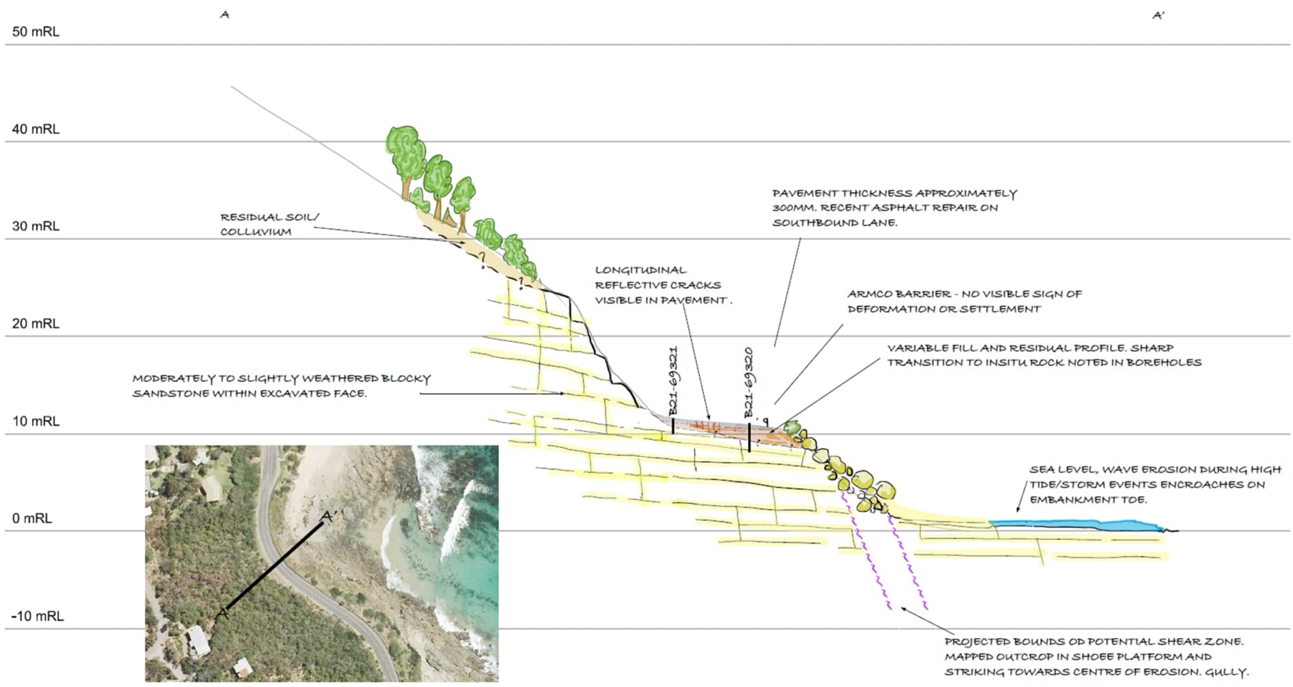


Figure 4 Cross section A-A'

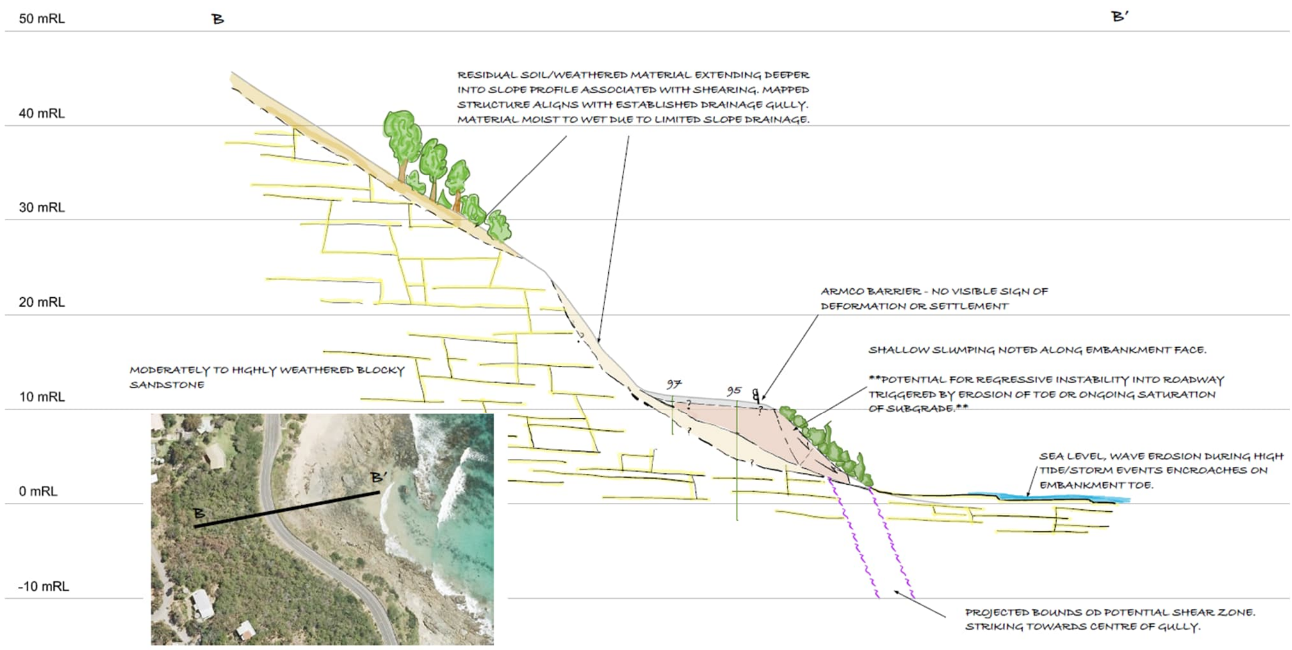


Figure 5: Cross section B-B'

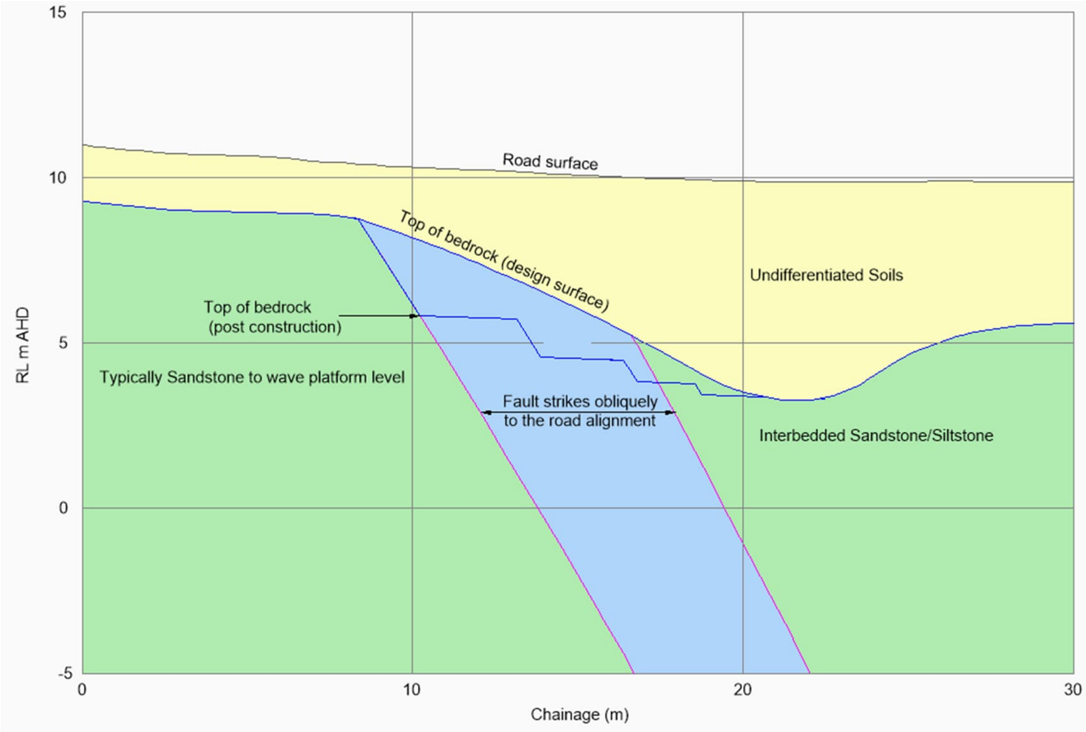
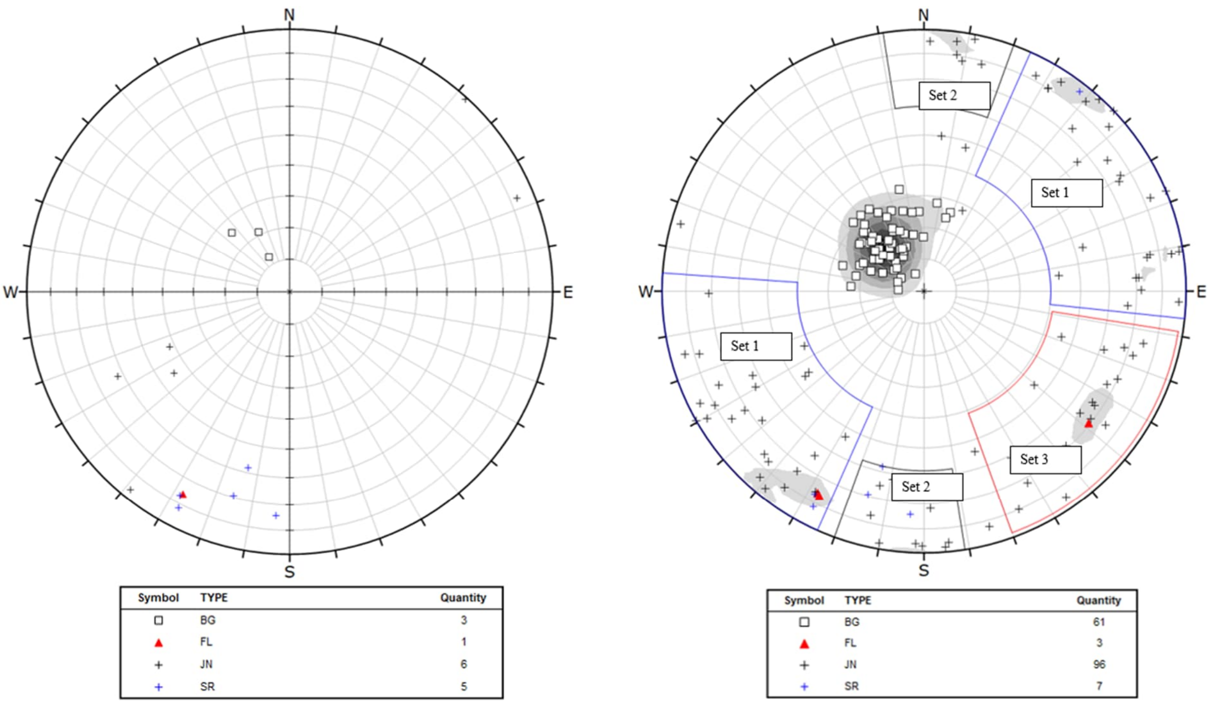


Figure 6: Long Section. Refer to Figure 3 (b) for Section location

2.3 STRUCTURE

Sandstone exposures within the cut face on the southern end of the site, Figure 2 (b) presented as a blocky rock mass which is consistent with mapping completed across the township. Mapping from available exposures and across the coastal exposures of Wye River and Separation Creek, and within similar domains, are presented in Figure 7 and Table 1.



(a) (b)

Figure 7 (a): Stereonet of features at the site mapped in 2021 (b) 2021 data supplemented with data from 2019 mapping at the site and the coastal exposures of Wye River and Separation Creek

2.3.1 Joints

Discontinuities are inferred to have formed during deposition (bedding) and post-depositional folding (joint sets). In folded sedimentary rocks, Fookes (2000) indicates that longitudinal, transverse joints and cross cutting joints are common. Two joint sets were mapped within the sandstone cut face on site and typically three sets are documented in the Wye River coastal exposures, Figure 7 (b) and Table 1.

2.3.2 Shears

Intact major shears and faults are seldom observed along the coast within the Otway Ranges due to wave and gully erosion removing the infill materials within the structure. Subsequently major geomorphological features (embayment, gully etc) form at the location of these structures.

A persistent fault was logged in 2019 as part of a separate investigation, Figures 7 (a) and 8. The author notes that the infill may include partially recemented boulders associated with Quaternary variations in sea level and erosion/deposition (Gill, 1979). The fault zone is inferred to have had a series of tightly spaced joints that were evacuated by erosion. This fault zone is inferred to control the abrupt change in depth to bedrock, Figures 3 to 6 and the location of the gully west of and above the road, Figure 3. Broadly the intersection of the fault with bedding (and bedding parallel shears) is inferred to control the location and axis of the gully.

On reconciling post construction records, the Point Sturt abandoned terrace 250 m southeast of the site was considered. This location was identified as of special geological significance (Rosengren, 1984) and has features including "cavitation weathering, abandoned channels and a level surface" approximately 7.5 m higher than current sea levels. On review of post construction records, Figure 6, in the authors view, a similar platform was likely present on the southern side of the gully. This is of significance to remediation of fill slopes along the GOR with slope aspects perpendicular to bedding dip direction as the subvertical nature of the edges of the platforms require special attention in investigation and design to make reasonable efforts to mitigate design variations during construction.

Table 1: Typical discontinuity sets – Wye River coastal exposures

Bedding Set	Joint Set 1 – Transverse	Joint Set 2 – Shear	Joint Set 3 - Longitudinal
Bedding	Joint strike orthogonal to bedding strike	Cross cutting bedding	Joint strike parallel to bedding strike
ORIENTATION (TRUE NORTH) DIP/DIP DIRECTION			
5 to 30/95 to 200 (20/145)	40 to 90/030 to 090 (70/050) 45 to 90/205 to 275 (75/245)	70 to 90/180 to 195 (80/190) 55 to 90/000 to 015 (75/005)	45 to 85/285 to 335 (70/300)
EFFECTIVE LENGTH (m)			
Dip slope and regional folds indicate 100's of metres	Horizontal 10 to 20m observed on rock platform Vertical generally less than 5m (limited by height of cuts and beds)		
EFFECTIVE SPACING (m)			
<0.25 to 0.75	Typically, 0.25 to 1 up to 5 m. Terminate at beds	Typically, 0.25 to 1 m, up to 5 m	
CONDITION			
Planar. Slightly rough to rough. Iron stained with clay veneers and seams common in highly weathered bedrock. Bedding parallel shears are less common (5%)	Highly weathered - typically Planar, slightly rough, iron stained with some clay seams and smooth surfaces. Moderately weathered or better - typically Planar, Rough, iron stained.		

(a) (b)

Figure 8 (a): Fault with re-cemented boulders/cobble infill which strikes sub-parallel to the gully. Location is highlighted in blue in Figure 3. (b) Looking along strike of the fault towards the GOR and Morley Avenue properties

2.4 MECHANISMS OF FAILURE

Two potential mechanisms of failure were identified: pavement settlement and translational sliding of the sideling fill slope.

The pavement settlement was considered likely to be controlled by one, or a combination of the following:

- Progressive collapse settlement of fill at the abrupt change in subgrade (shallow rock versus deep fill)
- Strain incompatibility between the two subgrade types (highly weathered rock versus saturated fill) including likely influence from the identified fault
- Saturated subgrade and insufficient depth of cover to the subgrade given the increase in heavy rigid vehicles post 2015 bushfires and re-building (i.e., pavement design related).

Regarding the translational slide mechanism, the buried contacts between the fill and residual soils and bedrock were gently dipping towards the east. As such, failure by translational sliding was governed by undercutting at the toe of the sideling fill slope and the overall slope of the fill (approximately at repose). Furthermore, due to the superelevation of the corner of the road, surface water was directed to the two landslide areas.

Kinematic failures in the rock mass were assessed to be barely credible. The intersecting planes from moderate dip joints of Set 1 intersecting and sub-vertical joints of Set 2 could cause small wedge failures however less than 1% of defect combinations supported this mechanism. Planar sliding on defects within the rock mass was not kinematically possible.

2.5 ROADSIDE HAZARDS

Two roadside hazards were assessed, and these include pavement settlement (Hazard 1) and translational sliding of the fill embankment (Hazard 2)

Hazard 1 was primarily associated with a vehicle impacting a stepped surface greater than 200 mm in depth. Note that creep along low angle defects dipping out of the slope was identified as a potential mechanism however it is unlikely to be the primary cause of pavement failure. This was supported by the Armco barrier that did not appear to be misaligned which otherwise would indicate the occurrence of mass movement.

Hazard 2 was a translational slide in the fill embankment. Regression of the landslide could result in a headscarp developing in the adjacent lane and vehicles may strike the resulting step in the pavement.

Mitigation of the hazards incorporated a combination of a cantilevered bored pile retaining wall with capping beam, inclusion of sub-surface cut off drains to the west of the roadway and improvements to kerb and channel with discharge of surface and sub-surface water further north where natural dune slopes were present along with a wide shoulder.

Although the hazards are not directly linked to structure in the rock mass, the bounds of the hazards were certainly controlled by structure. Post construction records indicated remnants of a buried shore platform from the last interglacial stage may have been present on the southern extent of the site where the buried geometry of bedrock was very steep to sub-vertical, Figure 6. The author later encountered a very similar issue at St Georges River, Lorne, Figure 1, and notes that geometry of the buried platform must be considered in scoping both geotechnical investigation and design.

3 CASE STUDY 2 – SKENES CREEK

At CH 1.2 km along Skenes Creek Road a thin wedge failure in sandstone bedrock occurred following heavy rainfall on 18 August 2018. The runout initially extended across both lanes and with most of the failure volume (volume in the order of 50 m^3) in the lane immediately adjacent to the slope, Figure 9. The landslide was cleaned up, but parasitic rock falls continued and between 10 to 11 August 2019 rockfall was documented by DTP to have reached the kerb of the opposing lane. Debris impact cracked the kerb. The site required a landslide risk assessment to DTP guidelines followed by conceptual to detailed design advice.

During the Author's initial visit in 2020, most of the cutting had thick vegetation, Figure 10. The slope at the 2018 wedge failure was comprised of weathered, blocky Sandstone. Two intersecting joint surfaces with relatively minor variation on southwest dip created a thin wedge, Figure 10. A limited zone was mapped between approximate project chainage Ch. 60 and Ch.150; however, it was clear that the joints controlling the failure were prevalent in all exposures. Evidence of several similar failures was observed in the cuttings, Figure 11. Due to the size of potential landslides and the limited space between the cutting and road, engineering controls would almost certainly be required to reduce the risks to target DTP levels, however vegetation clearance was required to map the exposures in detail. The extent of 2021 vegetation clearance was in the order of 180 m and the extent of works is highlighted in Figure 12.

Immediately prior to mobilisation for remediation of the cuttings, a further small landslide occurred, Figure 13. At this location up to 2m of colluvium was overlying a laminated sequence of slake affected siltstone and sandstone of very low strength and with significant shears parallel to dominant joint sets and bedding in this area. The lower portion of the failure was controlled by a wedge formed from the intersection of south dipping shears (55/180) with a shear zone dipping orthogonal to bedding (65/330), Figure 13. In a zone approximately 120 m long, no less than six similar landslides were inferred to have occurred between the widening of the road in the 1960s and 2022. For a frequency of landslide events of approximately 1 every 10 years, the assessed risk level exceeded typical target DTP levels of "low to moderate" risk or "no further action" required.

(a) (b)

Figure 9: Excerpt from DTP failure records from 18 August 2018 (a) Looking north (b) Looking east

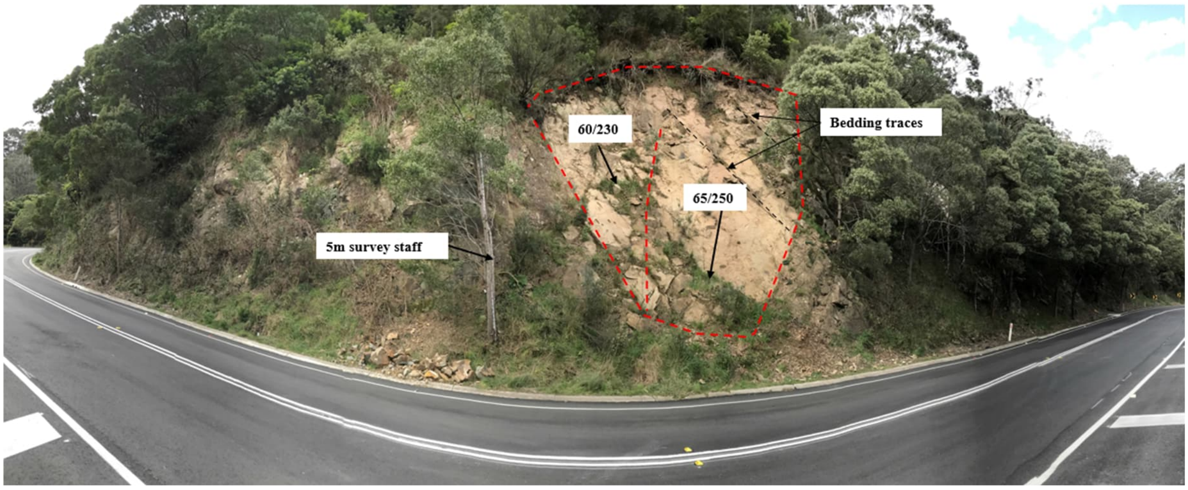


Figure 10: Wedge details in September 2020 looking east

(a) (b)

Figure 11: Planar slide at Ch. 115 of Figure 12. (a) Test bolting location and extent of failure (b) Sandy CLAY seam identified as the infill on the sliding plane

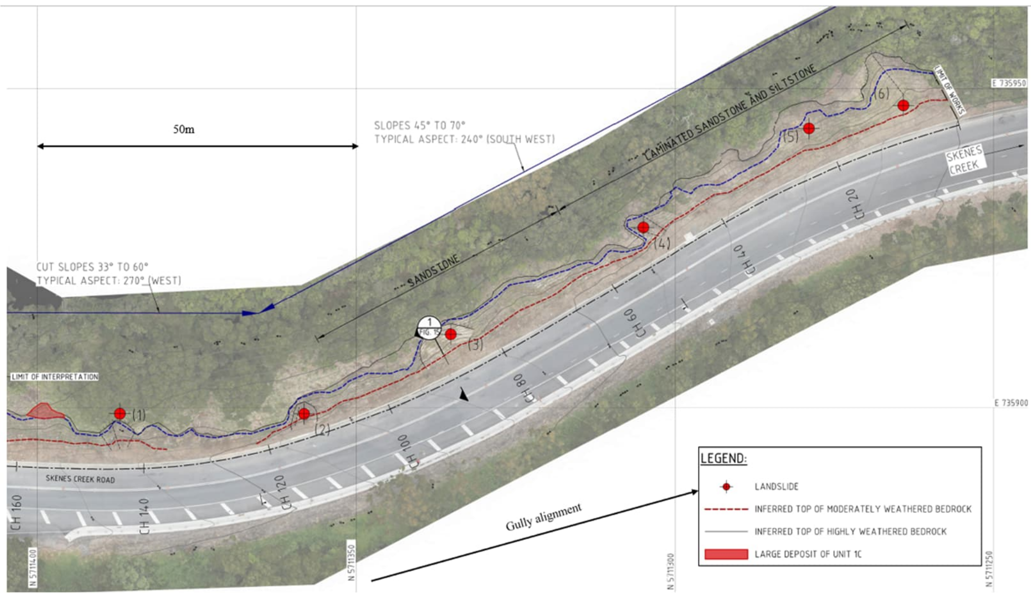


Figure 12: Extent of Skenes Creek site with project chainages highlighted

(a) (b)

Figure 13: Extent of landslide at location (5) of Figure 12 (a) left hand side in November 2021 (b) right hand side in June 2022

3.1 GEOLOGICAL SETTING

Cooney (1982) presents a readily available landslide inventory for the Skenes Creek and project area. The geology of Skenes Creek Road is well documented due to its proximity to major geological structures. For example, Cooney (1982) reported bedding typically dipping at 70 to 80° to the southeast with variation in dips inferred to occur due to proximity to the Skenes Creek Monocline, Figure 14. Furthermore, surface geology, local and regional structures in the vicinity of Skenes Creek were published by Medwell (1968), Cooney (1982), Duddy (1983), Geological Survey of Victoria (GSV), (1995) and Edwards (1996).

Various cross-sectional interpretations are available for the Skenes Creek folds (Medwell (1968) and GSV (1995) with differences in interpretation where a major regional fault is located. The varying interpretations along this section of coastline are not the purpose of this paper. The site is entirely within a region of sub-vertical to overturned beds with evidence of shearing that is universally referred to as the “Skenes Creek Monocline” by all the authors listed above. The author notes that the folding at the site could be readily interpreted as the southeast dipping limb of an asymmetrical anticline and the over-turned beds dipping at 85° towards the northwest may be aligned with major faults as suggested by GSV (1995). It is the Author’s opinion that the fold sequence from coast to hinterland aligns with Medwell (1968), Figure 14, and the reasons why Edwards (1996) has a different sequence is not clear. The published information highlights that complex structural conditions are to be expected at the site due to the close spacing of fold hinges, and steep bedding dips.

3.2 SITE INVESTIGATION

The extent of the site post vegetation clearing is highlighted in Figure 12. The site investigation was completed entirely with field mapping techniques that included:

- Detailed line mapping
- Mapping of defect dip/dip direction and persistence/spacing using a Diospatial photogrammetry model.

Typical excerpts from the photogrammetry-based mapping are highlighted in Figure 15. The mapping data from the site and photogrammetry mapping tasks was compared to Cooney (1982) to check for consistency with all data from line mapping and photogrammetry, Figure 16.

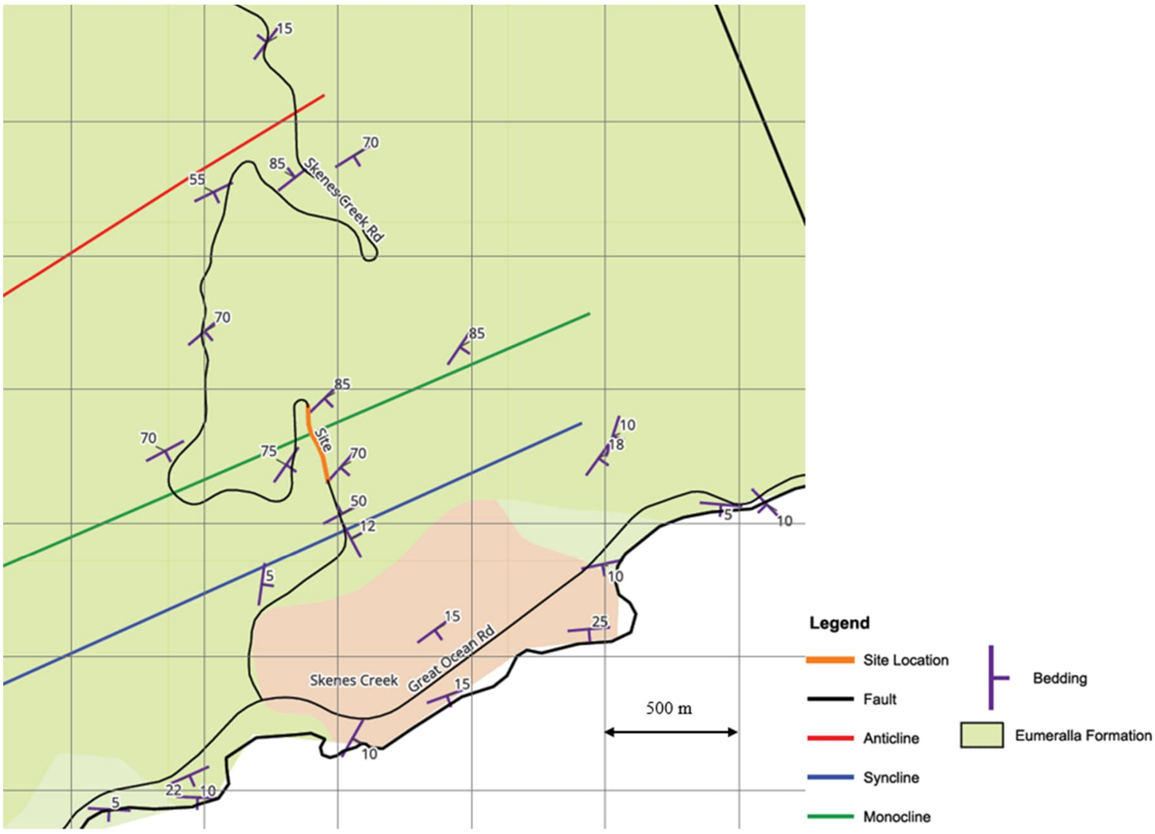


Figure 14: Regional geology with approximate published fold/fault locations based on Edwards et al, (1996), Cooney (1982), Medwell (1968) with minor variation on fold type from the Author

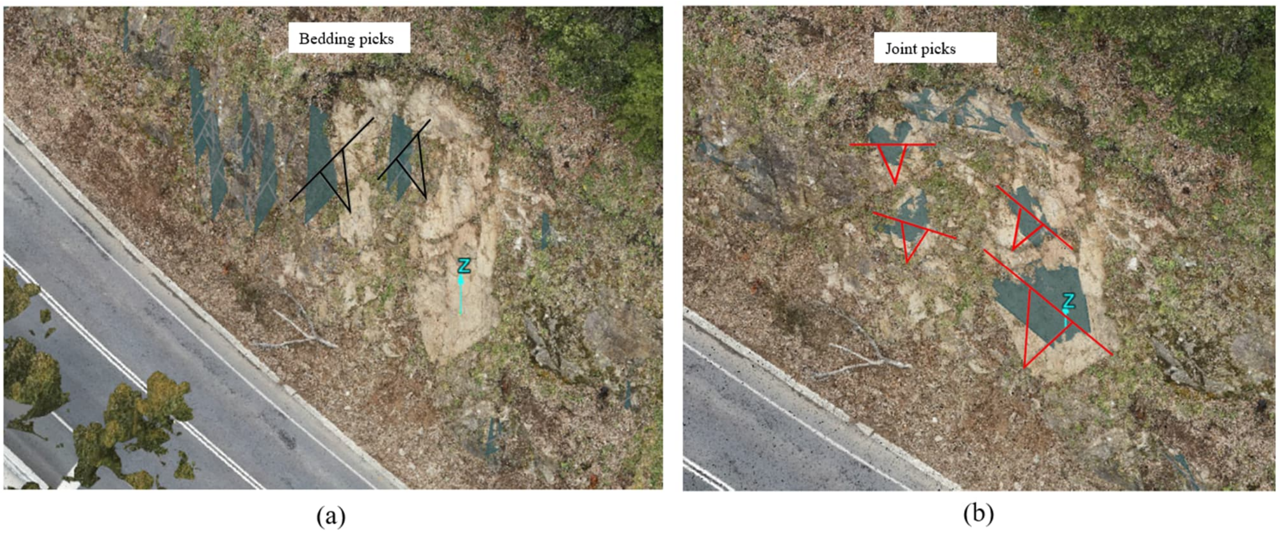
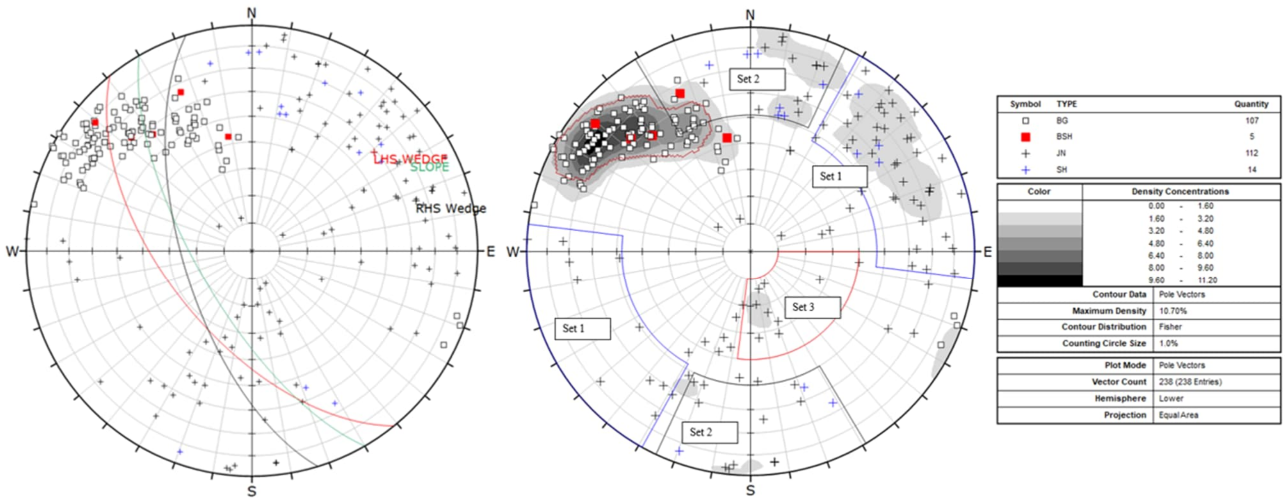


Figure 15: Photogrammetry picks at the Ch. 85 wedge failure (a) bedding planes (b) joint planes



(a) (b)

Figure 16 (a): Skenes Creek – all defects with 2018 wedge details (b): Skenes Creek – all joints and joint sets (note bedding and joints assessed independently)

3.3 GEOLOGICAL MODEL

The geological units identified in cut slopes in the assessment area included:

- Colluvium (UNIT 1C of Figure 17): A mixture of Sandy Clayey Silt/Sandy to Silty Clay with cobbles/boulders of Highly weathered Sandstone/Siltstone
- Residual (Unit 1R of Figure 17): Typically, a Clayey Sand
- Sandstone, fine to medium, very low to medium strength, highly to moderately weathered. Blocky rock mass with very steep bedding from 60 to 80° to the south east
- Laminated Siltstone/Sandstone, fine grained, Siltstone beds very low to low strength. Sandstone beds very low to medium strength. Highly to moderately weathered. Highly fractured rock mass with defect spacings of 50 to 100 mm common and rock mass impacted by slake.

The photogrammetry model allowed for inferred weathering changes to be mapped in detail, Figure 12.

3.4 STRUCTURAL MODEL

Table 1 presents a summary of the discontinuity sets. With regards to the bedding characteristics, it was noted that:

- Persistence of partings extended across the full height of the cut (up to 15 m in height)
- The spacing of bedding partings in Siltstone was typically less than 50 mm however the partings are difficult to differentiate due to slaking of the rock mass.

3.4.1 Joints

The evolution of the joint development is as per the Wye River case study albeit, significantly more evidence of shearing is observed at Skenes Creek. With regards to the mapped joints the three assigned joint sets are in good agreement with anticipated structure and gentle to moderate dip joints of Joint Set 3 are inferred to be underrepresented in the data set. For example, when comparing Figure 18 (a) and Figure 18 (b) these joints dip orthogonal to bedding and will vary in dip with bedding variations. i.e., a set that dips at 10 to 40° to the north west was anticipated.

3.4.2 Shears

Several shears and shear zones were mapped. Examples included bedding parallel shears Figure 18 (a), where bedding trends are highly variable within the shear zone, shears sub parallel to Joint Set 2, Figure 19 (a) and the closely spaced joints of Joint Set 1, Figure 19 (b).

Given the linear nature of the adjacent gully, Figure 12, it was inferred that the southwest dipping joints (Joint Set 1) may have relatively long persistence in the order of 10's to 100's metres which has contributed to the gully formation.

Similarly, due to the inferred presence of a significant fault, Section 3.1, it was considered likely that Joint Set 3 would have similar persistence.

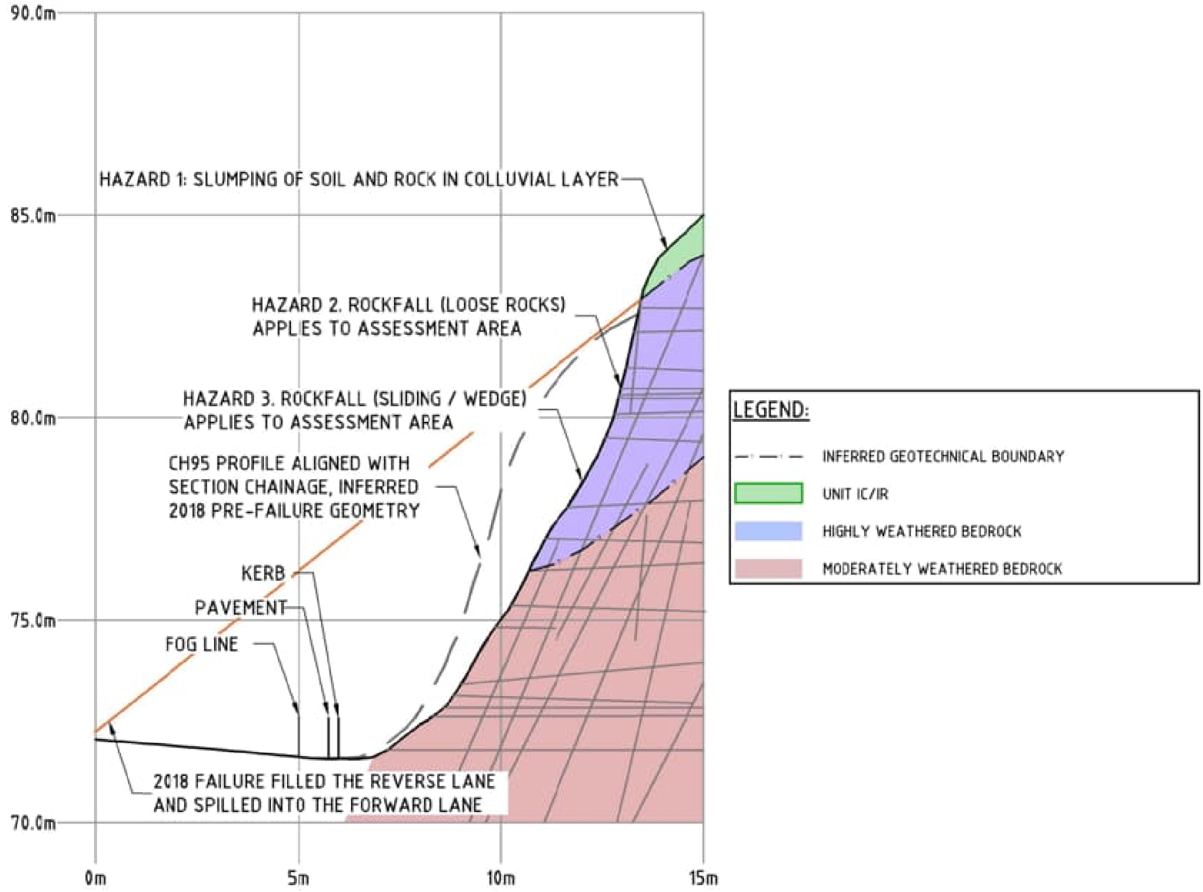


Figure 17: Section 1 with geological model and hazards

3.6 ROADSIDE HAZARDS

Three roadside hazards identified were broadly applicable to the entire cutting and involved a vehicle impacting debris at speed rather than the vehicle itself being struck by debris.

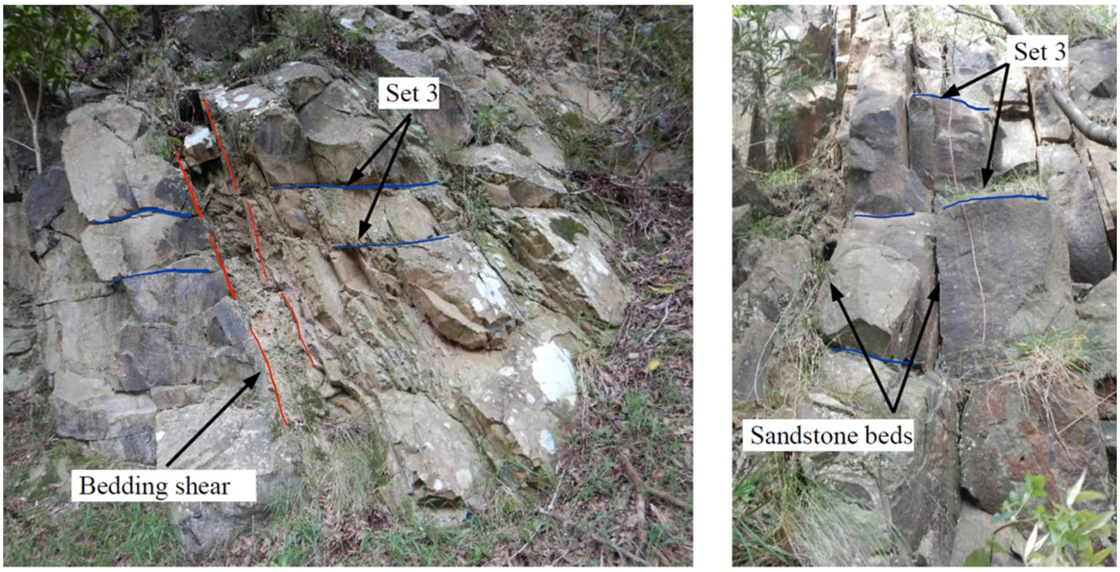
The typical block sizes from Table 2 are used to inform the likely size of blocks that may be impacted by a vehicle. A summary of the three hazards is presented in Figure 17 and includes vehicle impacting debris from the colluvial layer (Hazard 1), vehicle impacting debris from rockfall (Hazard 2) and a vehicle impacting debris from a much larger mass associated with planar/wedge failure in the rock mass (Hazard 3).

To mitigate the hazards, two domains were established with the boundary at the contact between the blocky Sandstone and in the interbedded Sandstone and Siltstone at Ch. 60, Figure 12. Both mesh drapery and spot bolting techniques were considered inappropriate due to the highly fractured nature of the rock mass and the observed failure volumes likely to damage drapery systems and require high maintenance.

Proprietary pattern bolt and mesh systems were the preferred DTP design solution and with the design considering the planar/wedge mechanisms as well as the capacity of the mesh to restrain colluvium, smaller blocks in the sandstone and slaking rock mass in the interbedded sequences. Testing of short bond lengths well in advance of construction validated the adopted bond strengths that were established based on field mapping estimates, Figure 11a.

Table 2: Typical discontinuity sets – Skenes Creek Road cutting

Bedding Set	Joint Set 1 – Transverse	Joint Set 2 – Shear	Joint Set 3 - Longitudinal
Bedding	Joint strike Orthogonal to bedding strike	Cross cutting bedding	Joint strike parallel to bedding strike
ORIENTATION (TRUE NORTH) DIP/DIP DIRECTION			
45 to 90/115 to 160 (70/140)	45 to 90/210 to 275 (65/240) 50 to 90/030 to 095 (65/060)	50 to 90/165 to 200 (60/180) 55 to 90/330 to 020 (70/000)	15 to 40/275 to 005 (25/330)
EFFECTIVE LENGTH (m)			
Regional folds indicate 100's of metres	3.0 to 10 to 15	3.0 to 10 to 15	0.5 to 10 to 15
	Observations limited by the height of the cut slope		
EFFECTIVE SPACING (m)			
0.3 to 3.3	<0.25 to 1	< 0.25 to 2	<0.25 to 3
Upper quartile of 1.2	Upper quartile of 0.75	Upper quartile of 0.75	Upper quartile of 1.5
CONDITION			
Typically planar, slightly rough to rough, iron-stained, parallel bedding shears with clay infill	Typically planar, slightly rough to rough, iron stained with some seams (clay and rock fragment infill) and smooth surfaces.		



(a) (b)

Figure 18: (a) Bedding shears and low angles joints at Ch. 60 (Set 3) (b) Sub-vertical sandstone beds at Ch. 147

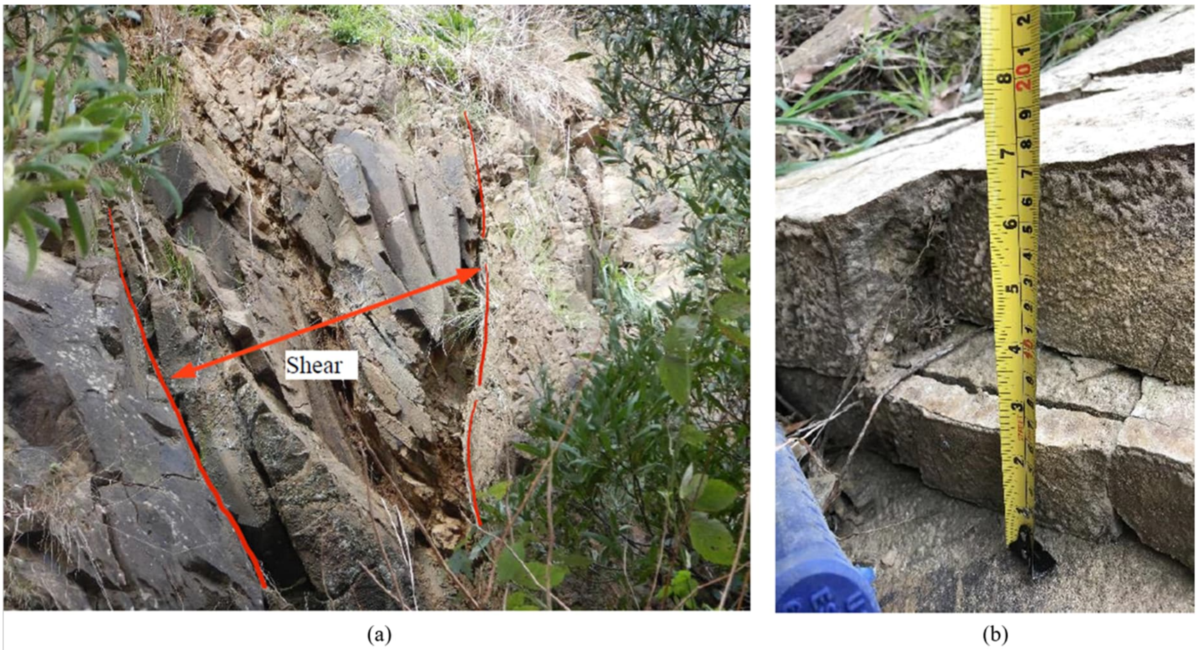


Figure 19: (a) Shears of Joint Set 2 at Ch. 149 (b) Lower bound joint spacing of Joint Set 1 at Ch. 85

4 SUMMARY

The two case studies presented illustrate the diversity of hazards in this terrain and the utility of combining local-scale field observation and range-scale geologic mapping to understand those hazards. Compiling engineering geological models in the Otway Ranges is well supported by a large database of valuable structural insights. A considerable proportion of the data is provided by the GSV and archived state publications.

The engineering geological model at Wye River assisted in avoiding overly conservative remediation. The structural model supported that the pavement was not likely subject to movement due to landslide but traditional pavement subgrade failure and that translational slide may occur north of the fault. Importantly, the fault controlled the geometry of both hazards. A learning post construction of the remedial solution was that a more concentrated effort in investigations to identify the edge of buried shore platforms beneath the side cast fills of the GOR may be warranted at similar sites.

The site at Skenes Creek presented unique challenges owing to the complexity of the regional structure. This site provides valuable insights into the hazards to road users on slope aspects cut perpendicular to the dip direction of bedding. These sorts of exposures are less common on the road network owing to the nature of the location of the GOR and the inland routes to the coastal towns.

Although mapped structure is relatively consistent across the shore platforms of Wye River and Separation Creek, the reader should exercise caution when referencing mapped structure at Skenes Creek. Owing to the likely presence of regional faults in that area, significant changes in structure are to be anticipated. That is structure within the established township of Skenes Creek itself may be significantly different.

5 ACKNOWLEGEMENTS

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CRediT authorship contribution statement

Dane Pope: Investigation, Formal analysis, Methodology, Writing - original draft.

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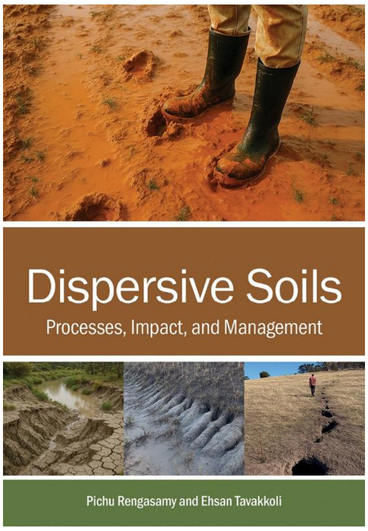
BOOK REVIEW

DISPERSIVE SOILS – PROCESSES, IMPACT AND MANAGEMENT

By: Pichu Rengasamy and Ehsan Tavakkoli

DOI: https://doi.org/10.56295/AGJ6129

When asked to review this book, I was pleased and keen to do so. My professional engineering geology background includes research into modified and stabilised clay soils, which means I have a good understanding of clay mineralogy, cement chemistry and soil science, including dispersive soils. Over the past 15 years or so I have dealt with dispersive soils in earthworks in QLD, specifically their identification and potential remediation as well as being involved from a client's perspective. However, I have tried to review "Dispersive Soils" from the perspective of a practising, generalist geotechnical engineer or engineering geologist without a deep understanding of soils science and dispersive soils chemistry.



Scope and structure

The book is split into a preface and eight chapters, each one with a comprehensive list of references.

The Preface contains the Authors' purpose of the book, which is "both a synthesis and a call to action", with the aim of resolving the "long-standing conundrum of sodic/dispersive soils - to clarify their chemical dynamics, connect these to their physical expressions and provide practical, sustainable remediation strategies'.

Chapter 1 clearly distinguishes between sodic soils and dispersive soils; sodic soil is defined as a soil with a high proportion of sodium ions relative to other cations on the soil's exchange sites (i.e. chemical), whereas a dispersive soil is defined by its behaviour when wetted (i.e. physical). The opening chapter focuses on this difference in detail and discuss the rationale for a dispersive, rather than sodic, soil focus. According to the authors the academic and professional soil community has increasingly embraced this change in recent years, and revised soil classification schemes now emphasise dispersion characteristics alongside or in place of traditional sodicity classes.

Chapter 2 outlines the key processes involved in the formation of dispersive soils beginning with an examination of natural and anthropogenic salinization processes. The chapter also introduces the physio chemical basis for clay dispersion, differentiates between dispersive saline and saline soils, and provides an overview of the global distribution and mapping limitations of dispersive soils with particular emphasis on Australian landscapes.

Chapter 3 discusses the soil chemistry principles relevant to clay dispersion from soil aggregates, the mechanisms of clay dispersion from soil aggregates, the physical behaviour of dispersive soils, and the impact on soil mechanics and engineering properties. The content of this chapter is most pertinent to the geotechnical professional even though the details are, understandably, "chemistry heavy". Discussions around exchangeable cations, the development and ionicity and covalency indices for clay cation bonds, the concept of dispersive potential, and the mechanisms of flocculation of soil aggregates in dispersive soils are covered in detail. Some mention is made of the impact of dispersive soils on soil shear strength and bearing capacity, in terms of shrink swell behaviour, foundations and road subgrades. The influence of dispersive soils on Atterberg limits is also briefly discussed.

Chapter 4 describes impacts on agriculture and management of dispersive soils. The text begins by stating dispersive soils represent a major challenge to sustainable agriculture landscape stability and environmental resilience across vast agricultural regions, and continues to talk about constraints to crop production, yield reduction and the principles of managing dispersive soils. A handy table (Table 4.1) which summarises key problems and management principles for dispersive and saline-dispersive soils across different pH conditions, and a second table summarising the key amelioration methods for managing dispersive and sodic soils, including their expected outcomes and practical limitations (Table 4.2), are provided. This chapter elaborates on chemical amendments to soils including gypsum, lime, organic amendments with chemical properties, calcium chloride and other salts, and emerging materials (e.g. phosogypsum, nano-gypsum, chelated calcium compounds and various others). Mechanical interventions, biological approaches and water management are also discussed.

Chapter 5, titled "Irrigation and Dispersive Soils", explores how irrigation practises contribute to the formation and intensification of dispersive soils, and critically examines the metrics and indices used to evaluate and manage irrigation water quality in such systems. This chapter, like Chapter 3, contains detailed descriptions of soil chemistry and cation exchange. Chapter 5 concludes that the quality of irrigation water is a critical determinant of soil structural stability and long-term productivity in irrigating systems.

Dispersive soils represent a significant challenge for environmental sustainability, ecological function, and resilience of civil infrastructure, so say the Authors, and this is discussed in Chapter 6 which focuses on the broader ecological, environmental, and infrastructural consequences of dispersive soils and considers both the mechanisms of degradation and possible avenues for mitigation. Descriptions of the mechanisms of various types of soil erosion are provided, including tunnel erosion, sinkholes, sheet erosion, rill and gully erosion, with some interesting black and white photos. Damage to civil engineering infrastructure is briefly discussed, including piping in earth dams, dispersive soils and landslides, quick clays and impacts to soil strength and bearing capacity. The chapter also briefly discuss the amelioration of soil dispersivity for engineering purposes.

Chapter 7 is titled "Identification of Dispersive Soils" and provides a comprehensive guide to identifying dispersive soils, beginning with traditional physical tests (the crumb, pinhole, and double hydrometer methods), followed by chemical indices (exchangeable sodium percentage (ESP), sodium adsorption ratio (SAR) as well as cation ratio of soil structural stability (CROSS) and its recent refinement, cationic charge ratio for soil structural stability (CROSSc). Emerging techniques and integrated approaches in dispersivity identification are also discussed including AI based predictive models, dielectric and geophysical assessments, remote and proximal sensing, and others. The main takeaway from the Authors, as a geotechnical practitioner, was that the identification of dispersive soils should not be constrained to isolated one-off tests (e.g. crumb tests). Instead, an evolving toolkit of laboratory, geospatial, and AI enabled methods should be used to support integrated multiscale diagnostics.

Chapter 8 outlines the critical knowledge gaps and innovation priorities that should be addressed to manage dispersive soils effectively. Building on the classification, mechanisms, and management strategies previously covered in the book, this final chapter moves from a national to a global perspective, drawing on recent advances in diagnostic tools, biological restoration, eco engineering and digital monitoring.

Overall Assessment

To conclude, I found the book "Dispersive Soils" by Pichu Rengasamy and Ehsan Tavakkoli to be useful and informative. In my opinion, the book's target audience are those operating in agricultural and soil science, rather than civil or geotechnical engineering. The important distinction between sodic and dispersive soils is clearly defined, and tables relating to key problems and management principles for dispersive soils may be of use to some. This book would be most useful for geotechnical practitioners with an interest in soil science and regular interaction with dispersive soils.

Bari Thomas

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- Text should be formatted to have 6 pt after paragraphs and after headings.
- No indent at the beginning of paragraphs.
- Title of Paper in 14 point Times New Roman, bold, uppercase, and centred in column.
- Main headings numbered 1, 2, 3…. etc. in 12 point Times New Roman, bold, upper-case and centred in the column.
- Sub-headings numbered 2.1, 2.2, 2.3 … etc. in 10 point Times New Roman, bold, upper-case and left justified.
- Minor headings numbered 2.1.1, 2.1.2 … etc. in 10 point Times New Roman, bold, lower-case and left justified
- Items in bulleted or numbered lists should not be separated by a

line, but should be indented by 10 mm.

- Formulae typed and numbered (1), (2), (3) … etc. and centred in the column.
- Captions for figures should be placed beneath the item and numbered Figure 1.
- Captions for tables should be placed above the item and numbered Table 1:
- Figures and tables should be referred to in the text as Figure 1, Table 1, etc.
- Figures and tables should be centred in the column.
- Do NOT use page numbers, these will be added later.
- In text citation according to the Harvard system of author (year) or (author, year) as appropriate. Multiple references should be separated by semicolons (author 1, year 1; author 2, year 2)
- References should be listed at the end of the paper in alphabetical order using the Harvard system: Author (year) title, publication, volume, pages, publisher with a 10 mm hanging indent and no blank line between each.
- Underlining should be avoided and symbols shown in italics.

FIGURES AND TABLES

All the journal is published in colour.

Where possible figures and tables should be placed at the correct position in the text. Figures should be imported into the document as a single image and not constructed in the word document. These should be sharp and of the correct size for incorporation into the finished document. The width of these must be less than or equal to the width of the text column (165 mm).

Where images are included in the paper they should be sent as a separate JPEG file to improve the picture resolution.

Photographs should preferably be good contrast gloss prints and of the correct size for incorporation directly into the copy. Please ensure that all such items are clearly marked to indicate position in paper.

EDITORIAL CONTACTS

The Editor is Hugo Acosta-Martinez, and the Editorial Panel consists of the Executive and State Chapter Representatives on the AGS National Committee.

The process of submission, peer review, discussion, re-submission, approval etc. of technical papers, is conducted using the peer review platform Scholastica. Technical papers should be submitted via:

Submit Manuscript button on the AGS Scholastica website:
https://ags.scholasticahq.com
or
Submit using Scholastica button on the AGS website:
https://geomechanics.org.au/journals/

Correspondence other than submission of, and queries about, technical papers, may be emailed to:

Editor, *Australian Geomechanics*
E-Mail editor@@geomechanics.org.au

ADVERTISING RATES

Every three months, *Australian Geomechanics* reaches more than 2500 professional geotechnical engineers and engineering geologists spread throughout Australia. Most of these are associated with significant site investigations, construction and computer analysis. So *Australian Geomechanics* provides a very targeted delivery for advertising.

Advertising rates include GST and from the 1st January 2020 are:

SIZE*	ONE ISSUE	TWO ISSUES	FOUR ISSUES
Cover Page	$1330	$1920	$3330
Full Page	$1030	$1760	$2640
Half Page	$530	$990	$1320
Quarter Page	$300	$480	$740

The prices quoted are for advertisements supplied in digital form as print resolution (240dpi or more) PDF, JPG or TIFF files.

* Files should be supplied at correct size with at least 3 mm bleed for designs that print to the edge of the page.

- **A4 Portrait –** Width: 210 mm x Height: 297 mm (+bleed)
- **Half A4 Landscape –** Width: 210 mm x Height: 148.5 mm (+bleed)
- **Half A4 Column –** Width: 105 mm x Height: 297 mm (+bleed)
- **Quarter A4 Column –** Width: 105 mm x Height: 148.5 mm (+bleed)

Inserts into an individual mail-out of *Australian Geomechanics* can be accepted at a minimum charge of $1330 (including GST).

Advertising queries should be addressed to:

Sara Lanesman, Email: *lanesman@optusnet.com.au*

ADVERTISEMENT DESIGN

If required AGS can arrange the design of adverts for *Australian Geomechanics*. The advertiser shall provide logo (high resolution), heading, text content, other images (photos) and style guide (if advertiser has one), otherwise styles and colours will be made similar to company's website styles or other provided media.

AGS will provide 1-2 design options and allow for two revisions of the chosen concept.

AD SIZE	APPROX. MAXIMUM WORD COUNT	COST
Full Page	250	$350
Half Page	150	$250
Quarter Page	75	$150

If design is required, material should be submitted no later than the first business day of February for the March issue, May for June, August for September and November for the December issue.