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Sustainable Geotechnics in Design, Materials, Construction and Maintenance

PROCEEDINGS OF THE 2023 AGS ANNUAL SYMPOSIUM

HELD ON

November 10th 2023

Organising Committee: M. Tamadon, C. Rujikiatkamjorn, S. Zargarbashi, H. Khabbaz, A. Parsa, A. Clark, S. Mirlatifi and AHMK Zaman

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Australian Geomechanics Society

Geomechanics is the application of engineering principles to the earth sciences to improve continually the accuracy, efficiency, cost-effectiveness and safety of construction projects both above and below ground, including the recovery of the earth's mineral resources. It remains an imprecise discipline due to the infinite variety of conditions in the earth's crust but correspondingly offers a fascinating and rewarding field of research and practice.

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 to the International society of Soil Mechanics and Geotechnical Engineering (ISSMGE), the International Society for Rock mechanics (ISRM) and the International Association for Engineering Geology and the Environment (IAEG).

SCHEDULE:

08:00-08:45: Registration
08:45-09:00: Opening address

Session 1: Sustainable Geotechnics in Design, Session Chair: Mehdi Tamadon (GHD)

09:00-09:35: Keynote lecture -Prof Nilo Consoli (Federal University of Rio Grande): Reuse of Iron Exploitation Waste as A New Binder for Tailings Stabilization in Dry Stacks: Circular Economy Approach

09:35-09:55: Ms Jessica Dalton (WSP): Case Study Comparing Embodied Carbon Emissions in Two Road-Over-Rail Bridge Foundation Designs

09:55-10:15: Mr Jonathan Cheng (Arup): Embodied Carbon Assessment of Geotechnical Works

10:15-10:35: Q&A

10:35-11:05: Morning tea

Session 2: Sustainable Geotechnics in Construction, Session Chair: Jennifer Hambling (Transport for NSW)

11:05-11:30: Invited Presentation-Mr James Ng (Kypreos Group): Sydney's First Roads Made with Recycled Coffee Cups

11:30-11:50: Mr Jason Hellmuth (GHD): Sustainable choices in geotechnics – a case study of quarry to parkland conversion

11:50-12:10: Dr Saman Zargarbashi (WSP): Opportunities for Sustainable Geotechnical Engineering Practice: Two Case Studies from Australia

12:10-12:30: Q&A

12:30-13:30: Lunch

Session 3: Sustainable Geotechnics in Material, Session Chair: Anastasia Suchowerska (SMEC)

13:30-14:05: Keynote lecture -D/Prof Buddhima Indraratna (University Technology - Transport Research Centre): Innovative Use of Recycled Rubber Elements for Sustainable Rail Infrastructure

14:05-14:25: Prof Ernesto Villaescusa (Curtin University): Investigation of a calcinated clay product for shotcrete support

14:25-14:45: Dr Asal Bidarmaghz (University of New South Wales): Synergising Sustainability in Geotechnical Engineering - Key Insights and Future Prospects

14:45-15:00: Q&A

15:00-15:30: Afternoon tea

Session 4: Sustainable Geotechnics in Maintenance, Session Chair: Alice Clark (AURECON)

15:30-15:55: Invited Presentation-Prof Hadi Khabbaz (University Technology - Transport Research Centre): Exploring Impacts of Abundantly Available Sustainable By-Product Materials in Australia on Stabilizing Expansive Soils

15:55-16:15: Mr Adnan Sahyouni (Menard): Ground Improvement Sustainable by Nature

16:15-16:35: Dr Hamid Mortazavi Bak (Beca): Study on Shear Response of Biopolymer-MICP Treated Soil-Steel Interfaces

16:35-16:50: Q&A

16:50-17:10: Closing remarks

17:10-17:45: Networking and Drinks

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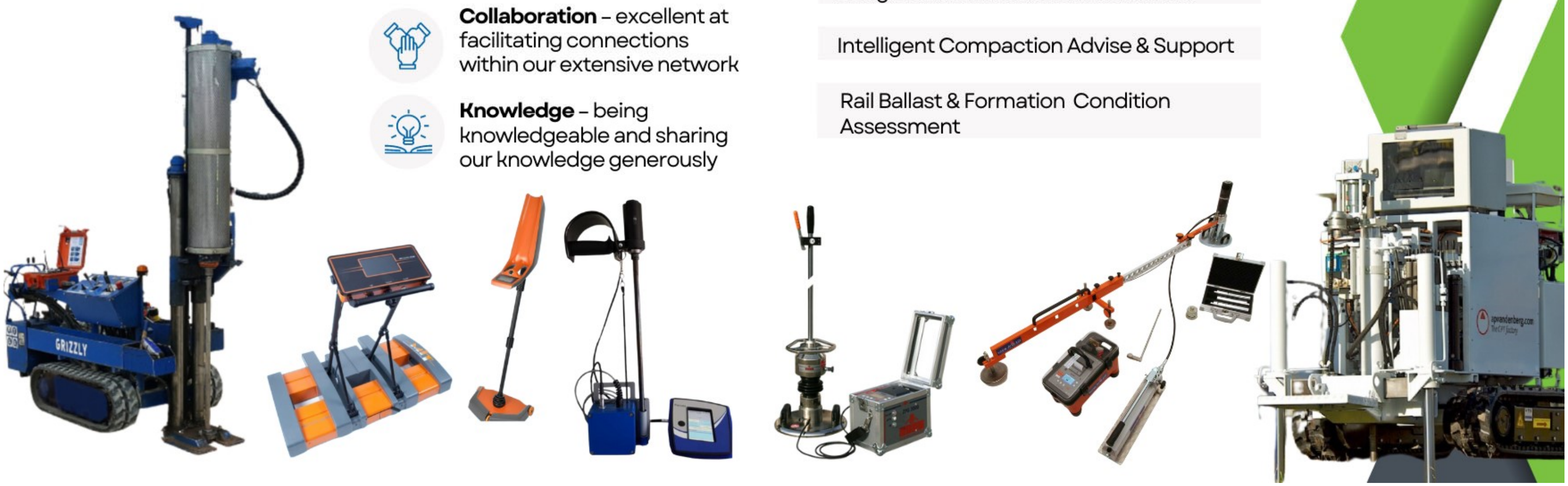


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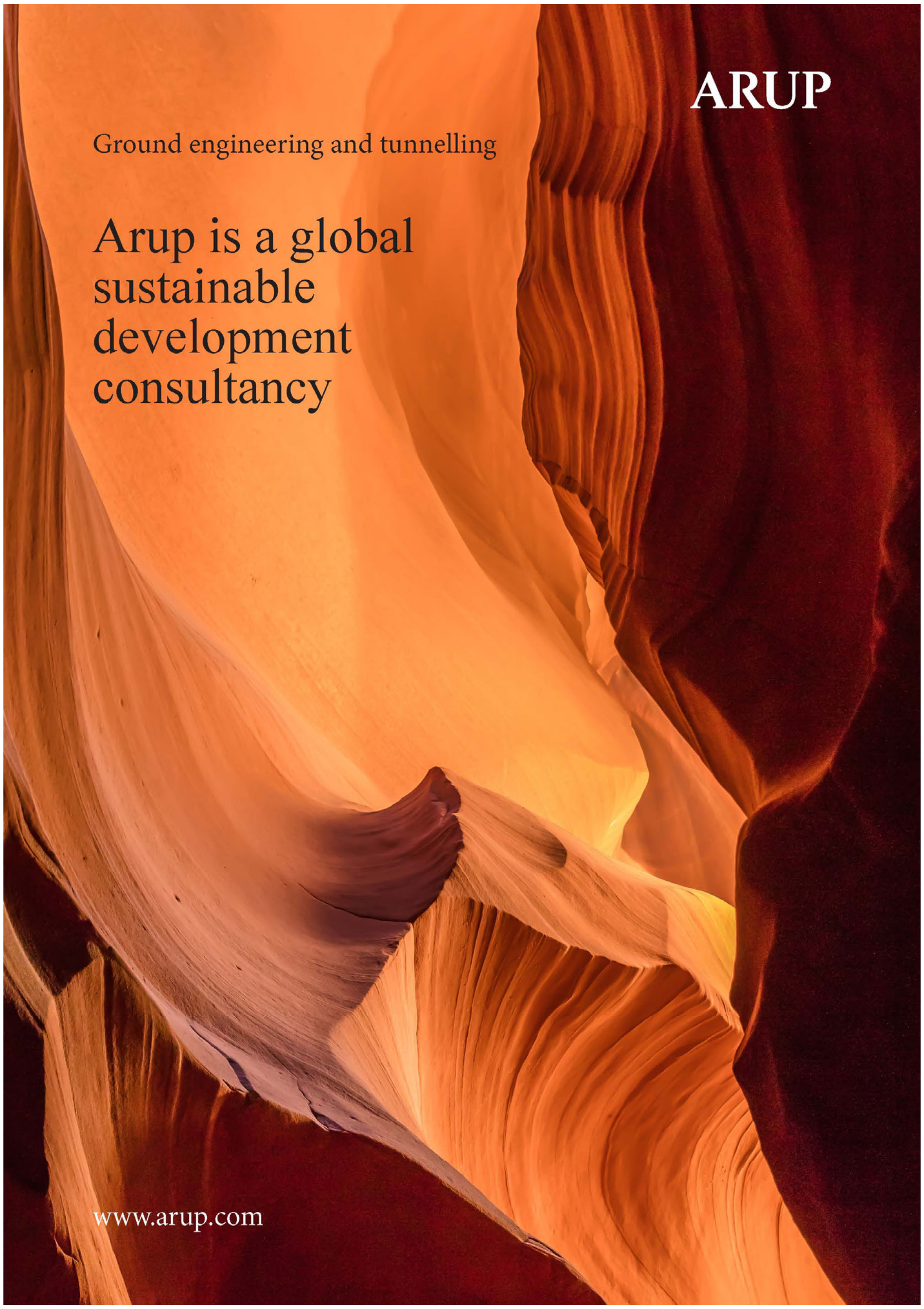
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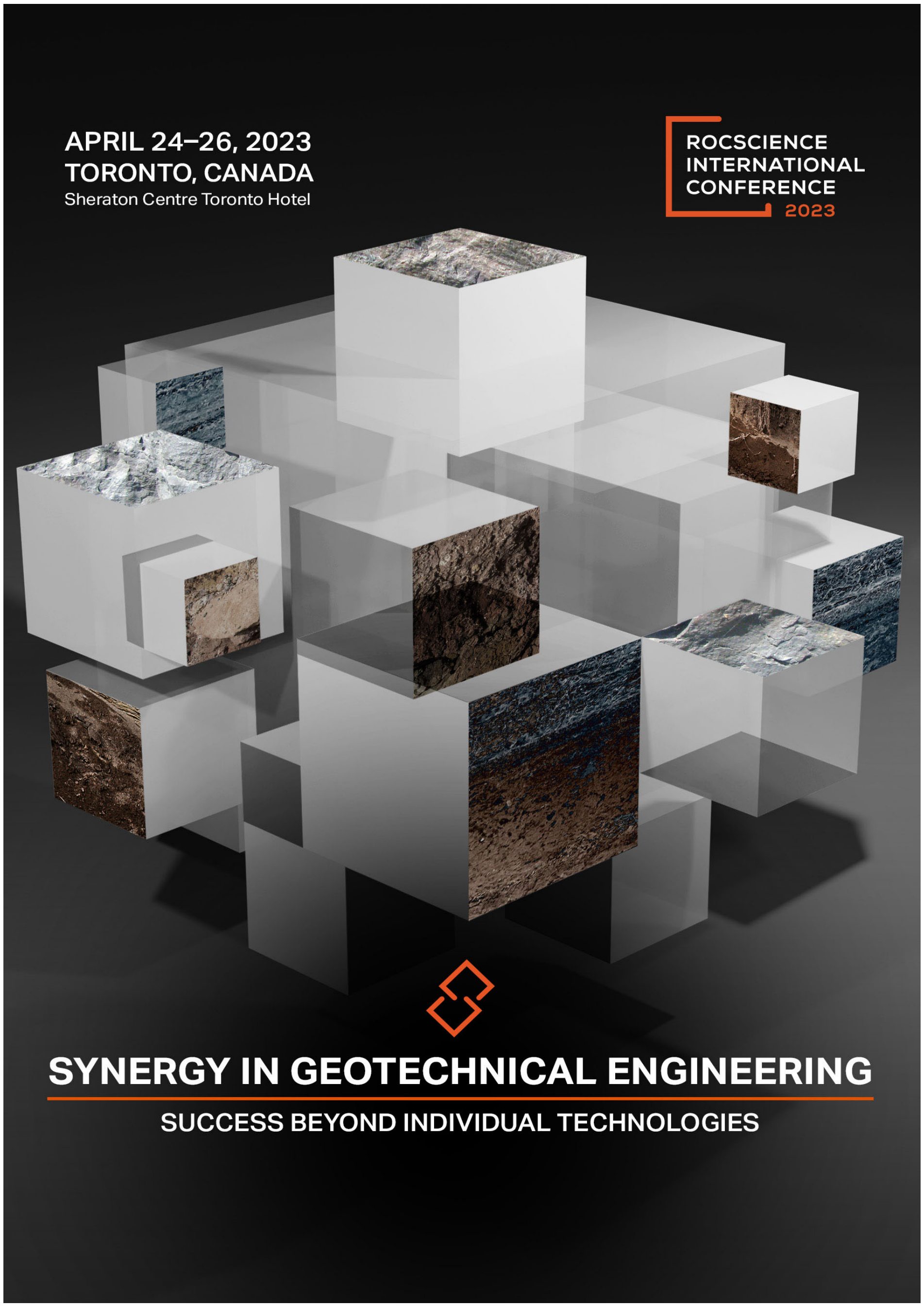
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Sustainable Geotechnics in Design, Materials, Construction and Maintenance

PREFACE

This document contains the accepted papers submitted and peer-reviewed for the 27th annual symposium organised by the Sydney Chapter of the Australian Geomechanics Society (AGS). The symposium presents state-of-the-art practices, new research findings and case histories that demonstrate sustainable design, construction and development. The papers on sustainable geotechnics cover various aspects of design, materials, maintenance and construction techniques. The symposium aims to keep practicing geotechnical engineers, engineering geologists, and other engineering professionals informed of recent developments in this field. It also recognises the need to gather the experience of those practicing locally and internationally and facilitate the transfer of knowledge and sharing of their experiences. These symposia continue to be one of the successful platforms for bringing together key stakeholders of the Australian geological and geotechnical community. It also provides a premier platform for participants to present and discuss innovations, trends and concerns, as well as practical challenges encountered, and solutions adopted in the field.

This symposium is the cooperative effort of many authors and qualified reviewers. The editors and organising committee wish to thank the authors, who have generously contributed their time to prepare the various papers and the colleagues of the authors, who assisted with time, secretarial, drafting and other facilities. Appreciation is also extended to our sponsors for supporting the Australian Geomechanics and Geological community.

Mehdi Tamadon, Cholachat Rujikiatkamjorn, Saman Zargarbashi, Hadi Khabbaz, Ali Parsa, Alice Clark, Sam Mirlatifi and AHMK Zaman

On behalf of the AGS Symposium Organising Committee,
The Australian Geomechanics Society, Sydney Chapter

Annual Seminars of AGS Sydney Chapter

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1	1997	Pavement Design Beyond 2000	A Leventhal
2	1998	Recent Developments in Piling Practice in Sydney	P Andrews
3	1999	Flexible Retaining Walls: Design to Prevent Failure	P Andrews and P Hewitt
4	2000	Computer Methods	P Hewitt and J Carter
	2001	Excavation Retention	T Walker and P Hewitt
6	2002	Landslide Risk Management	B Walker and T Walker
7	2003	Geotechnical Instrumentation and Construction Works Compliance Testing	G Scholey and T Walker
8	2004	The Engineering Geology of the Sydney Region – Revisited	G Scholey, M Parmar, G Young and G McNally
9	2005	Geotechnical Aspects of Tunnelling	H Buys and T Gourlay
10	2006	Soft Ground Engineering	H Buys, R Moyle and P Hewitt
11	2007	Engineering Advances in Earthworks	R Moyle, R Lindbeck and H Liu
12	2008	Foundation Design and Construction	R Moyle, R Lamont and B Ewers
13	2009	Geosynthetics – New Materials for Modern Infrastructure	B Ewers, H Buys and H Liu
14	2010	Seismic Engineering- Design for Management of Geohazards	C Rujikiatkamjorn, J McIlveen, R Lamont and M Haysler
15	2011	Coastal and Marine Geotechnics- Foundations for Trade	C Rujikiatkamjorn, J McIlveen, G Blumberg, J Smith and C Y Tey
16	2012	Advances in Geotechnical Aspects of Roads and Railways	H Khabbaz, C Y Tey, O Stahlhut and C Rujikiatkamjorn
17	2013	Retaining Structures: Recent Advances and Past Experiences	H Khabbaz, C Rujikiatkamjorn, M van Uden, C McColgan and S Mirlatifi
18	2014	Resilient Geotechnics	H Khabbaz, C Rujikiatkamjorn, S Mirlatifi, C McColgan and M van Uden
19	2015	Recent Developments and Experiences with Groundwater and Excavation	C Rujikiatkamjorn, S Mirlatifi, Katarina David, A. Hulskamp, and M van Uden
20	2016	Ground Stabilisation Techniques for Problematic Soils and Unstable Rocks	C Rujikiatkamjorn, S Mirlatifi, A. Hulskamp, T Mcwilliam and H Khabbaz
21	2017	Geotechnical innovations and challenges for transport infrastructure	S Mirlatifi, , C Rujikiatkamjorn, G Vorobieff, D Airey, A Parsa, A Hulskamp, H Khabbaz, H Buys and K Oj,
22	2018	Advances in Site Investigations, Monitoring and Instrumentation	A. Parsa, H. Khabbaz, S. Mirlatifi, C. Rujikiatkamjorn, A. Hulskamp, AHMK Zaman, and P. Rajarathnam
23	2019	Innovations in Geotechnical Construction and Design	A. Parsa, C. Rujikiatkamjorn, S. Mirlatifi, H. Khabbaz, A. Hulskamp, AHMK Zaman, P. Rajarathnam, M. Baringa, and T. Gourlay
24	2020	Geotechnical Advances and Challenges in Urban Development	A. Parsa, C. Rujikiatkamjorn, S. Mirlatifi, H. Khabbaz, A. Hulskamp, AHMK Zaman, P. Rajarathnam, K. Kianfar
25	2021	Geotechnical Lessons Learnt – Building and Transport Infrastructure Projects	A. Parsa, M. Tamadon, C. Rujikiatkamjorn, S. Mirlatifi, H. Khabbaz, A. Hulskamp, AHMK Zaman, P. Rajarathnam, K. Kianfar, S. Zargarbashi
26	2022	Reliability-based Design: Advances, Innovation and Experiences	P. Rajarathnam, A. Parsa, M. Tamadon, C. Rujikiatkamjorn, S. Zargarbashi, H. Khabbaz, S.Mirlatifi, AHMK Zaman, A. Hulskamp
27	2023	Sustainable Geotechnics in Design, Materials, Construction and Maintenance	M. Tamadon, C. Rujikiatkamjorn, S. Zargarbashi, H. Khabbaz, A. Parsa, A. Clark, S. Mirlatifi and AHMK Zaman

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All papers have been peer reviewed in accordance with the AGS Symposium review procedure.

REUSE OF IRON EXPLOITATION WASTE AS A NEW BINDER FOR TAILINGS STABILISATION IN DRY STACKS: CIRCULAR ECONOMY APPROACH

Nilo Cesar Consoli[1]; João Vítor de Azambuja Carvalho[1]; Alexia Cindy Wagner[1]; and Rodrigo Beck Saldanha[1]
[1]*Graduate Program in Civil Engineering, Universidade Federal do Rio Grande do Sul, Porto Alegre, RS, 90035-190, Brazil.*

ABSTRACT

The dried allocation of the tailings, rather than the disposal in a slurry form, appears as an alternative to attend new legislation and improve the safety of mines operation. Also, the use of cementing agents in dry stacking facilities can enhance aspects of operations such as guaranteeing dilatant behaviour at the base and increasing tailings' strength. The present research assesses the technical and environmental viability of a new alkali-activated cement (AAC) in iron ore tailings stabilization. The mechanical response of compacted tailings-AAC specimens was evaluated through strength and shear modulus tests while Life Cycle Assessment (LCA) was performed to verify the sustainability of this new binder when compared to conventional AAC. This new binder is derived from the residues of iron exploitation and is intended for use in new disposal schemes, such as dry stacks. The AAC is mainly composed of metakaolin (MK), produced from the residual soil removed during the mining activity, and sodium silicate (SS), produced with sandy tailings. Using tailings and waste in AAC production aligns with sustainable practices, minimizing resource consumption and promoting waste recovery. Also, LCA demonstrates a lower impact for tailings AAC when compared to conventional AAC. In addition to environmental and mechanical aspects, using this AAC supports the application of circular economy in mining since it enables the reuse of waste produced in mine operation as a substitute for conventional cement (that involves another industry and raw materials).

1 INTRODUCTION

Due to recurring disasters, agencies in Brazil have introduced new regulatory frameworks prohibiting the construction of upstream tailings dams and requiring the closure of existing ones built with such a technique (Schaper et al., 2020). Kossoff et al. (2014) present several environmental impacts of tailings dam failures from immediate (e.g., deaths of animals and humans) to medium and longer-term impacts (e.g., soil and water contamination). According to the authors, dam failures have economic consequences related to environmental damage, cleanup of the affected area, and interruption in the mining and processing operations.

One alternative solution to attend to the novel regulations is the dried allocation of the tailings rather than slurry disposal. Stacked filtered tailings disposal is a methodology that has emerged alongside advances in dewatering technologies in recent decades (Gomes et al., 2016). Consoli et al. (2022) have presented the trend of disposing filtered tailings in stacks by considering Portland Cement-Iron Ore tailings mixtures. However, Ordinary Portland Cement (OPC) has been over-consumed in the last decades (Ahmed et al., 2021), and it is costly economically and environmentally, although being an efficient and trustworthy binder. Thus, it is necessary to find new cementitious materials to substitute the OPC.

Lately, alkali-activated cements (AAC) have gained notoriety as an OPC replacement (e.g. Corrêa-Silva et al., 2019; Lotero et al., 2021; Rivera, Castro, et al., 2021; Rivera, Coelho, et al., 2021). In essence, AAC results from the association of a precursor, usually a material with silico-aluminates in amorphous phases, and an alkaline solution as an activator. Then, the reaction of the activator and precursor forms a new material in a metastable state. This reaction product develops cementitious properties due to the combination of alkali-aluminosilicate and/or alkali earth-aluminosilicate phases through complex chemical interactions (Duxson et al., 2007).

The usage of AAC in the mining industry offers the opportunity to incorporate waste from the mining process as precursors and activators. This fact favours the transition of the mining industry to a more circular chain of production. The definition of the circular economy (CE) concept is not static, and can be broad, involving several principles and proposals (Merli et al., 2018). Still, in summary, it can be understood as the need to overcome the usual produce-use-waste linear thinking and look at ways of bringing value to materials considered waste (Lèbre et al., 2017; Upadhyay et al., 2021). Although there is no consensus over CE definition, some characteristics are recurrent on the definitions, such as fewer pollutant emissions, more recycling, environment-friendly disposal, reasonable utilization of resources, reducing waste, and maintaining the value of resources as long as possible (European Commission, 2017; Merli et al., 2018; Zhao et al., 2012).

Accordingly, this paper proposes using a novel Alkaline Activated Cement (AAC) produced from mining waste with Iron Ore Tailings (IOT) for cement-bonded dry stacking purposes. Therefore, unconfined compressive strength and ultrasonic pulse velocity tests (UPV) were carried out for compacted tailings-AAC specimens to evaluate their strength and stiffness response, respectively. A full factorial design setting was used considering four curing periods, five amounts of cement and three dry unit weights. Strength and stiffness results were correlated to the porosity/binder content index – η/B_{iv} (Consoli, Winter, et al., 2018). The new binder environmental impacts are evaluated through life cycle assessment (LCA) and compared to those of conventional AAC. Thus, the aim of this paper is to verify the technical and environmental feasibility of using an alternative ACC in dry stacking of IOT.

2 CIRCULAR ECONOMY IN MINING: CHALLENGES AND OPPORTUNITIES

The mining industry is one of the larger waste producers in the world. The waste management in this sector is usually linear (produce-use-waste), and there is little effort to establish sustainable production that accounts for waste reuse (Lèbre et al., 2017; Zhao et al., 2012). Thence, practices more aligned with CE can be valuable and applicable to this industry segment. The valorisation of tailings could contribute towards the adoption of a circular economy in mining, although this now confronts several challenges and bottlenecks in economic, technological, and environmental terms (Kinnunen & Kaksonen, 2019; Upadhyay et al., 2021).

Recycling is a relevant process within the circular economy. Besides limiting waste production, the reuse and recycling of mining wastes reduce the use of natural resources and favour the local economy. Reusing solid mining wastes and mining process waters also contributes to reducing the exposure of humans and the environment to contaminated products (Lottermoser, 2011). Nevertheless, the existing recycling methods in mining need further development. The recycling flowsheets are individual, and it is necessary to redesign them considering economic issues, tailings properties, and possible environmental impacts (Edraki et al., 2014).

2.1 PROCESSING AND DISPOSAL METHODS

Mining operations produce a wide range of waste which can be separated mainly into waste and tailings (Spitz & Trudinger, 2019). According to Blight (2010), the mine wastes are usually stored separately. The soil portion is stacked for future uses and the rocks are placed in dumps (with or without compaction) or are used as landfills. Already the tailings produced from ore exploration are found in a slurry form due the water use in process (Fig. 1) which favours their deposition in large impoundments as tailings' dams (Kossoff et al., 2014; Xiaolong et al., 2021). The tailings in the structures are routinely found in a loose and saturated state, which brings additional risks to this method of deposition (Islam & Murakami, 2021; Yao et al., 2021). Furthermore, slurry deposition form has unpleasant disadvantages, such as high embankment costs, extensive water handling, and difficulty rehabilitating the slimes (Spitz & Trudinger, 2019).

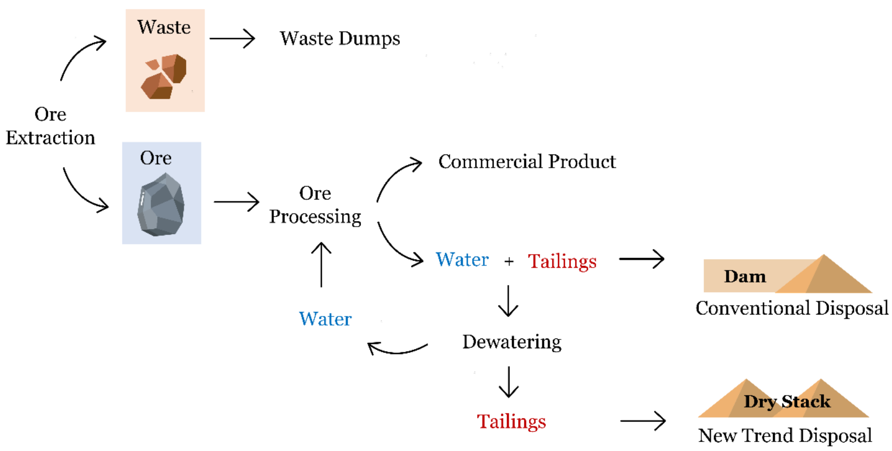


Figure 1: Comparison of disposal methods of tailings

One alternative method that follows the advances in dewatering technologies is the disposal of tailings by stacking (Consoli et al., 2022; Davies, 2011). In this method, the tailings are filtered to low moisture contents and disposed of in a compacted form in stacks that can achieve heights of around 200 to 300m (Lupo & Hall, 2011). Once such methodology involves stacking dewatered tailings, the need to dispose of this material in dams is eliminated, which implies a more harmonious use of the landform. Moreover, the risks associated with the material's static and/or dynamic liquefaction are substantially diminished compared to the conditions encountered in traditional tailings dams (Chang et al., 2011; Hu et al., 2017; W. Li et al., 2018).

Water is part of the ore beneficiation process (Fig. 1). According to (Davies, 2011), the future of mining requires increasing attention to water use worldwide, mainly due to the large amount of water lost in conventional dam tailings by seep and evaporation. Furnell et al. (2022) suggest that dewatering techniques allow recovering water to reuse in ore beneficiation plants. The maximum water recovery may be achieved with filtered tailings. Also, water recovery usually offers a cost-benefit that offsets the capital and operating cost of the filtration plants (Davies, 2011).

The compacted tailings must attend prerequisites for stacking use, which depend on the projected use and geometric configuration. In this method the tailings are compacted at different compaction degrees within the stacking zones. At the borders (structural zones), the material is usually deposited in dense states, while it is deposited at lower compaction degree (or even loose) at the centre (non-structural zones) (Lupo & Hall, 2011). One fundamental aspect is the guarantee of dilatant behaviour under drained shearing at stacking base. The mixing of tailings with a cementitious compound, such as ordinary Portland cement (OPC), prior to the tailings stacking can contribute to achieving this condition (Consoli et al., 2022), reducing or even mitigating liquefaction potential.

2.2 ALTERNATIVE CEMENT ADDITIONS TO ENHANCE DRY STACKING

The use of OPC has proven to be a promising solution for the dry stacking of tailings (Consoli et al., 2022). Nonetheless, substantial amounts of energy and natural resources are demanded in its production, causing the emission of high quantities of carbon dioxide (CO_2) and other greenhouse effect gases (Çankaya & Pekey, 2019; Gálvez-Martos et al., 2021; M. Li et al., 2020). These side effects and the mining industry's requirements for environment-friendly solutions reinforce the idea of the new alternative search.

The AAC is recently being considered as a replacement for OPC, including solutions for tailings (Servi et al., 2022). These binders are characterized by a mixture of an alkaline activator and a precursor (a source of silica and aluminium in amorphous phases). After mixing these materials, a chain of reactions that produces a new stable phase with bonding properties develops (P. Krivenko, 2017).

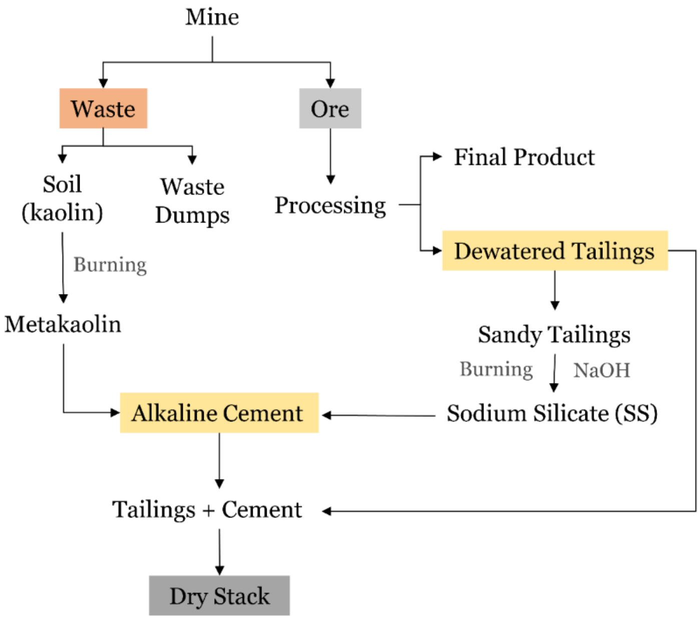
One advantage of AACs is that various materials can be employed in their production. Thus, industrial by-products and construction wastes have already been tested for AACs production. The most regularly investigated precursors have been blast furnace slag, fly ash from coal combustion, and metakaolin, whereas sodium-based and potassium-based hydroxides and silicates have been the routinely employed activators (Fernández-Jiménez et al., 2017; Palomo et al., 2014; Zhang et al., 2021).

Regarding the activator choice, those that provide soluble silica, e.g. sodium silicate (SS), promote faster reaction rates than alkaline hydroxides (Lima et al., 2021; Torres-Carrasco & Puertas, 2017). However, the SS synthesis performed commercially demands high energy consumption and generates CO_2 in large quantities, being a bottleneck for reducing environmental impact of AACs (Dal Pozzo et al., 2019; Labianca et al., 2022).

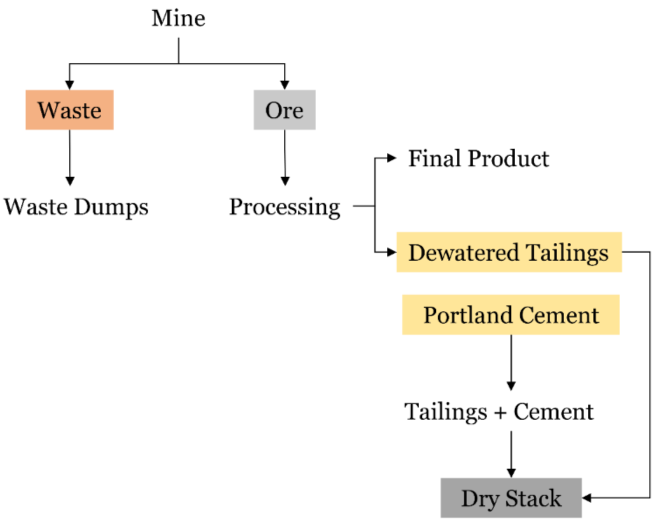
Different materials have been used as substitute sources of silica for producing sodium silicate, such as rice husk ash (Andreola et al., 2020; Lima et al., 2021), waste glass (Tchakouté et al., 2016; Torres-Carrasco et al., 2014) bamboo leaves (Kow et al., 2014), silica fume (Bernal et al., 2012), among others. They have shown great feasibility in producing economically competitive and environment friendly AACs compared to conventional cements. In this regard, Vogt & Lameiras (2019) defined an innovative procedure to produce SS powder from sandy tailings obtained by the flotation process in the iron ore beneficiation plant. The procedure consists of removing ultrafine fraction from the tailings (particles smaller than 40 μm) and reducing moisture content through filtering, centrifugation, and drying processes (105 °C). Finally, the resulting material reacts with sodium hydroxide in a furnace at temperatures between 400 and 500 °C.

One commercially adopted solution for the precursor is metakaolin (MK), which is not usually considered a residue or by-product. However, iron ore exploitation involves excavating and removing soil, which can be rich in different minerals, including kaolinite. These soils are usually treated as waste and disposed of in waste dumps. Nonetheless, this material can give rise to MK after being calcinated at high temperatures (approximately 800°C). The use of the MK originated from waste in conjunction with an also waste-based SS, giving rise to a new AAC fully integrated into the production chain of iron ore exploitation.

Figure 2 compares the production chain considering the OPC versus the AAC addition to the tailings before disposal in the dry stack. It is noted that the AAC adoption reduces the disposal of tailings and soil waste by reusing these materials within the same production chain, transforming both in commercial products. It also avoids using other natural resources or raw materials (as in the OPC industry). Furthermore, this AAC could provide means for implementing the circular economy practices in mining operations.



(a)



(b)

Figure 2: Production chain considering dry stacking deposition of (a) alkali-activated cement-iron ore tailings mixtures and (b) Portland cement-iron ore tailings mixtures

2.3 ENVIRONMENTAL SUSTAINABILITY

According to Nagaraj (2010), mineral processing research efforts are likely to prioritize economic outcomes. For Edraki et al. (2014), it is necessary to think primarily about reducing environmental problems in the long term and not only about the current cost benefits. In this regard, Zhao et al. (2012) divided the reuse of waste and tailings into two main perspectives: the reprocessing for further extraction of valuable ore that is still present in the materials, and the production of new substances, mainly as building materials, using waste as a raw material.

Concerns about environmental sustainability are directly correlated with the reuse of iron mining tailings. Basically, the tailings sent for storage in ponds have the potential to be used as raw material in the production of concrete (Che et al., 2019; Huang et al., 2013; Zhu et al., 2015), ceramic production (Das et al., 2000; Yellishetty et al., 2008) and in the pigment industry (Pereira & Bernardin, 2012), interlocking blocks (Filho et al., 2017; Ravi Kumar et al., 2012) among other processes that require granular aggregates, trace elements of iron and sources of silicates. The substitution of natural resources by tailings/waste meets the precepts established in the Brundtland Commission (Brundtland, 1987), formed under the supervision of the United Nations, which proposes a rational use of natural resources enabling that the needs of future generations are not compromised. However, a large portion of the tailings from the iron mining process still do not have a significant alternative destination that can avoid the disposal of this material in dams. In this case, the dry storage process with an alternative binder is a necessary path in the search for an environmentally friendly storage process that enables its future use as a resource source.

In this context, the production of sodium silicate and metakaolin with mining tailings to produce an alkali-activated cement, in the stabilization of the mining tailings themselves, is a technical alternative that avoids the use of natural resources, reduces the generation of tailings promoting environmental, economic and operational benefits. In the specific context of products that incorporate waste, Life Cycle Assessment has a fundamental role in identifying and quantifying environmental impacts in different categories, such as consumption of natural resources, greenhouse gas emissions, air, soil and water pollution, among others. This detailed analysis provides valuable information to guide decision-making in the selection of more sustainable materials and processes (Ouellet-Plamondon & Habert, 2015).

3 EXPERIMENTAL PROGRAM

3.1 MATERIALS CHARACTERIZATION

3.1.1 Iron Ore Tailings

The IOT were collected from a beneficiation plant in the Quadrilátero Ferrífero (Iron Quadrangle) located in Minas Gerais (MG), Brazil. The characterization for the IOT was carried out following the determinations and standards for geotechnical materials, as they would constitute the base material to be improved for the deposition in stacks.

The physical characteristics of the IOT were evaluated through the particle size distribution (ASTM, 2021a), the Atterberg limits (ASTM, 2017), and the specific gravity (ASTM, 2014). The compaction characteristics of the IOT were assessed using standard and modified efforts in agreement with ASTM standards (ASTM, 2021b, 2021c). Table 1 summarizes the main physical properties of the IOT.

Table 1: Physical characteristics of the iron ore tailings

Physical properties	Iron ore tailings	Test method
Liquid limit (%)	-	ASTM D4318
Plastic limit (%)	-	
Plastic index (%)	non-plastic	
Specific gravity	2.92	ASTM D854
Coarse Sand (2.00 mm < diameter < 4.75 mm) (%)	0	ASTM D7928
Medium Sand (0.425 mm < diameter < 2.00 mm) (%)	4.0	
Fine Sand (0.075 mm < diameter < 0.425 mm) (%)	49.0	
Silt (0.002 < diameter < 0.075 mm) (%)	42.0	
Clay (diameter < 0.002 mm) (%)	5.0	
Effective diameter (D_{10}) (mm)	0.0085	
Maximum dry unit weight at standard effort (kN/m^3)	19.2 (w = 11.2%)	ASTM D698
Maximum dry unit weight at modified effort (kN/m^3)	20.6 (w = 9.2%)	ASTM D1557

The chemical and mineralogical characteristics of the IOT were also evaluated. From X-ray fluorescence (XRF) analyses was determined that the major components present in IOTs (Table 2) were silicon (69.6%), iron (24.1%), and aluminium

(4.8%) which agrees with the crystalline peaks of quartz (SiO_2), hematite (Fe_2O_3) and kaolinite ($Al_2Si_2O_5(OH)_4$), identified in the diffractogram of Fig. 3.

Table 2: Chemical composition (wt.%) of the iron ore tailings (IOT), metakaolin (MK) and sodium silicate (SS) obtained through X-ray fluorescence (XRF) analysis

% wt	SiO_2	Al_2O_3	Fe_2O	MnO	K_2O	CaO	Na_2O	SO_3	P_2O_5	Others	LOI [a]
IOT	69.6	4.8	24.1	0.4	0.15	< 0.1	< 0.1	0.1	< 0.1	0.15	0.2
MK	48.9	40.8	0.28	0.31	0.36	2.7	< 0.1	Nd [b]	0.13	0.6	< 0.1
SS	43.9	< 0.1	4.6	< 0.1	< 0.1	1.1	49.7	Nd [b]	< 0.1	0.2	0.12

[a] LOI: loss on ignition.
[b] Nd: Not detected.

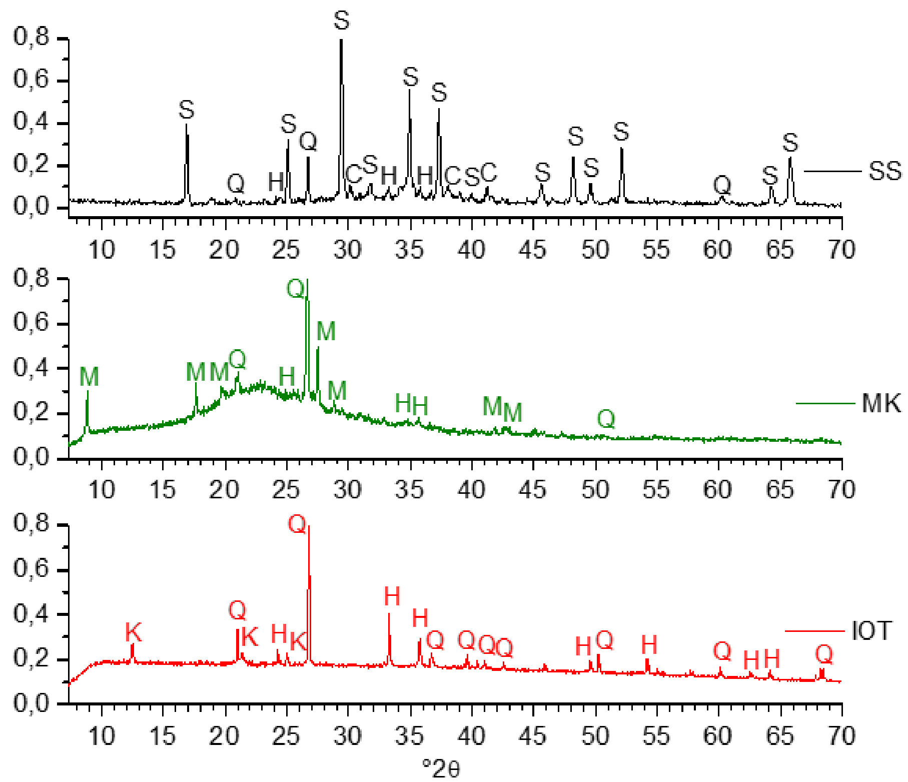


Figure 3: X-ray diffraction (XRD) analysis of the starting materials. S: Sodium silicate (Na_2SiO_3); Q: Quartz (SiO_2); C: Sodium carbonate (Na_2CO_3); H: hematite (Fe_2O_3); M: muscovite ($KAl_2(AlSi_3O_{10})(OH)_2$); K: kaolinite ($Al_2Si_2O_5(OH)_4$)

3.1.2 Cement Components

The AAC is essentially made up of metakaolin (MK) and sodium silicate (SS). These two materials are originated from iron extraction plants and produced according to the methodology described in (Vogt & Lameiras, 2019). For them, it was performed chemical and mineralogical characterization.

The MK is originated from the waste soil above the ore deposit before exploration. The waste soil was initially investigated to confirm the presence of kaolinite and was calcinated at elevated temperatures (approximately 800° C) to produce the MK. The XRF tests results attested to the presence of the following major elements: silicon (48.9%), aluminium (40.8%) and calcium (2.7%) for MK (Table 2). Mineralogically, XRD analyses reveal a typical pattern of a semi-crystalline material, composed of a pronounced halo between 15° - 35° of 2θ angle (Fig. 3), usually associated with the reactive alumino-silicates present in the vitreous phase (Consoli et al., 2021; Lotero et al., 2021; Palomo et al., 1999), with slightly pronounced peaks about the presence of quartz (SiO_2), muscovite ($KAl_2(AlSi_3O_{10})(OH)_2$) and hematite ($Fe_2O_3$) minerals.

Regarding the SS, the XRF tests (Table 2) indicate the existence of sodium (49.7%), silicon (43.9%), iron (4.6%), and other trace elements in small amounts (< 0.1%). In addition, XRD test results confirm, by the high intensity of the characteristic peaks in Fig. 3, that the SS is mainly composed of sodium silicate (Na_2SiO_3) and quartz (SiO_2) crystals with small amounts of sodium carbonate (Na_2CO_3) and hematite (Fe_2O_3). The specific gravity of the solid components of the AAC was 3.1 g/cm3, determined by the Le-Chatelier flask method (ASTM, 2016a).

3.2 GEOTECHNICAL TESTS

A fully crossed-design setting (Montgomery, 2013) was used for the strength and stiffness tests. Then, concerning the dosages, four curing periods (4, 7, 28, and 90 days), three different dry unit weight values (17, 18, and 19 kN/m³), and five amounts of AAC (1, 2, 3, 4, and 5%) were tested. Therefore, 15 dosages were evaluated in triplicates, totalling 45 specimens for each of the four curing periods.

The increase in strength and stiffness over time in the mixtures was evaluated by adopting different curing periods. The chosen dry unit weight values were based on the compaction results (Tab. 1), and a single moulding moisture content (w) of 11% was established regardless of the adopted γ_d (e.g. Consoli et al., 2007, 2020; Corrêa et al., 2021; Festugato et al., 2021). The choice of AAC contents (up to 5%) was associated with economic issues and optimization. Higher strength could be achieved by the balance of compaction and binder content. Furthermore, small amounts of cement are considered to avoid difficulties for possible reprocessing tailings which may be of interest as new technologies develop.

3.2.1 Specimens Moulding and Curing

Cylindrical specimens (50 mm in diameter and 100 mm in height) were moulded using the undercompaction method (Ladd, 1978). The moulding process was initiated by the weighing and mixing the dry materials (IOT and AAC) until visual uniformity was obtained. The AAC was composed, in mass, of 62% of metakaolin and 38% of sodium silicate (Vogt, 2022). The amount of cement (C) was calculated over the mass of dry tailings, and the defined quantities follow the practice of tailings stabilization with OPC (e.g. Consoli et al., 2017; Consoli, Winter, et al., 2018; Festugato et al., 2013). Right after, distilled water was added, and the mixing progressed until a mass having a uniform consistency was formed. Then, the specimen was statically compacted in three layers inside a cylindrical split mould until reaching the target dry density. Following, the specimen was retrieved from the mould and cured in a humid room. The procedure adopted for constructing the dry stacks in the field would be very similar to that performed for the specimens. In this case, the AAC would be mixed with the filtered tailings (at the target moisture content) and then compacted in layers as in landfills and pavements where soil improvement is used.

3.2.2 Unconfined Compressive Strength Tests

The unconfined compressive strength tests were performed on an automatic loading press coupled to a 10 kN load cell (0.005 kN of resolution). A displacement rate of 1.14 mm per minute was used. Each specimen was submerged in a water tank for 24 hours before the test to minimize possible suction effects on the strength (e.g. Consoli et al., 2007). It established an acceptance criterion that the individual strength value (q_u) should not deviate by more than 10% of the triplicate's mean strength, considering the same mix design.

3.2.3 Ultrasonic Pulse Velocity Tests

The initial shear modulus (G_0) can be calculated as the product between the square of the shear wave velocity (V_s) and the apparent density (ρ) of the media for elastic isotropic media (ASTM, 2016b). In this sense, the G_0 of the compacted iron ore tailings-AAC specimens was assessed using an ultrasonic pulse velocity (UPV) device responsible for emitting shear waves at a constant frequency of 150 kHz. The shear wave velocity (V_s) was determined by measuring the travel time of these waves across the test specimens using transducers coupled to the ends of the samples. A special coupler gel was used to attach the transducers to the specimen. Since the UPV is a non-destructive test, it was performed on the identical specimens that would be submitted to the unconfined compression tests.

3.3 LIFE CYCLE ASSESSMENT

The purpose of conducting a Life Cycle Assessment was to assess the environmental impacts of two distinct methods for stabilizing tailings and to determine the suitability of these solutions within the context of a circular economy (Angelis-Dimakis et al., 2016; Biganzoli et al., 2018; Laso et al., 2018; Sassanelli et al., 2019). Using tailings as raw materials for AAC has an impact in reuse and recycling of materials, key principles in CE concept. Thus, the LCA was used to verify the performance of a system with reuse and recycle and identify the possible benefits in relation to commercially available solutions. The assessment focused on a functional unit of 100 m³ of stabilized tailings with 4% of AAC. The first method involved using alkali-activation stabilization with alternative production of SS and alternative MK (using tailings as raw materials), while the second method used alkali-activation stabilization with commercial metakaolin and sodium silicate (known as conventional alkali-activated cement, AAC). The life cycle assessment (LCA) involved conducting a cradle-to-gate analysis of the production process for the binder, which consisted of sodium silicate and metakaolin. The system studied in Fig. 4 accounted for all emissions related to the energy used in production and the raw materials involved. The commercial production technique for sodium silicate is through the hydrothermal process which involves dissolving a silica source with sodium hydroxide inside a reactor at elevated pressure and temperatures (8.15×10^3 MJ/tonne of SS). For the production of sodium silicate from mining tailings, the technique developed by (Vogt & Lameiras, 2019) is proposed. The process consists of heating the tailings with sodium hydroxide (1:1) in a furnace at a temperature between 400°C and 500°C (2×10^3 MJ/tonne of SS).

Metakaolin used in commercial production is obtained by prospecting kaolinitic clays and subjected to calcination. This calcination process alters the structure of the clay inducing chemical and physical changes in the generation of metakaolin. However, when producing metakaolin using mining waste, the conventional clay extraction operation is eliminated. Instead, clay from the overburden is utilized as a substitute for the conventional raw material. In the present work the impact assessment did not take into account the mixing process of sodium silicate, metakaolin, tailings, and water for the reaction and compaction processes in tailings stabilization. This is because the mixing process was identical for both binders and therefore did not contribute to any discernible differences in the impact assessment.

Secondary inventory data was collected from the Ecoinvent 3.8 database (Wernet et al., 2016) using SimaPro software. The quantity of each inventory is presented in Figure X flowcharts of the evaluated systems. The impact assessment was conducted using CML2001 method (Guinee, 2001). The assessment included the following impact categories: abiotic depletion (AD), global warming (GW), ozone layer depletion (OLD), human toxicity potential (HTP), freshwater aquatic ecotoxicity potential (FAETP), marine aquatic ecotoxicity potential (MAETP), terrestrial ecotoxicity potential (TAETP), photochemical oxidation (PHO), acidification (AC), and eutrophication (EU). Additionally, the Cumulative Energy Demand - CED was incorporated into the evaluation of the impact assessment.

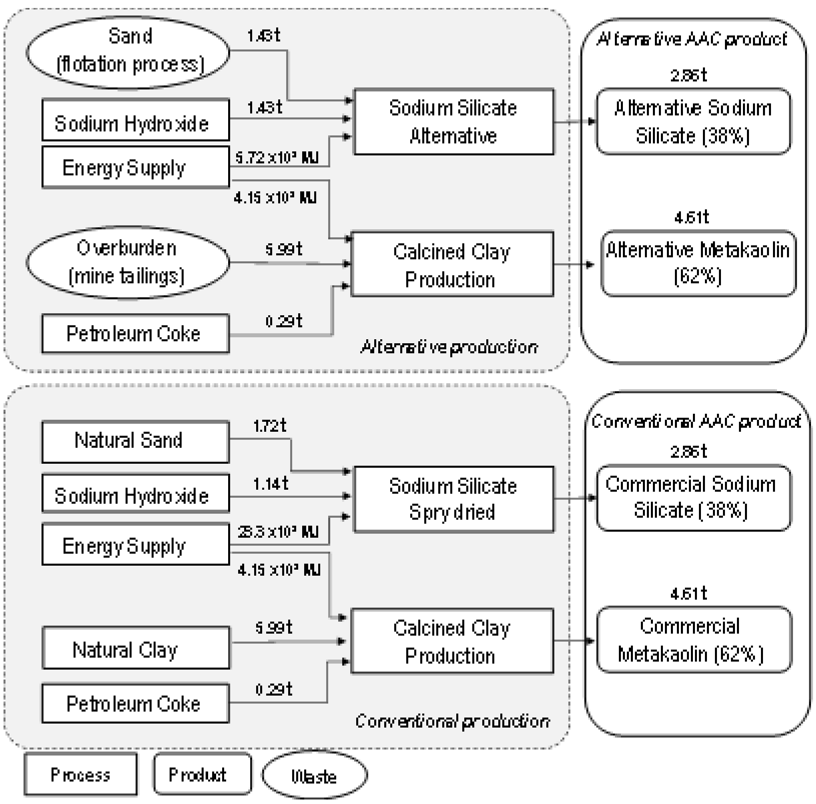


Figure 4: System boundaries and processes for the production of alkali-activated cement for stabilization of 100 m³ of tailings

4 RESULTS AND DISCUSSION

The specimens moulded using 1% of the AAC disintegrated when submerged (after each curing period), indicating that such an amount of cement could not generate an adequate cementing matrix considering the studied curing periods (Consoli et al., 2007). The same has occurred for the specimens of 2% of the AAC cured along 4, 7, and 28 days. For 90 days of curing, it was possible to prepare specimens. However, this period is too long to be considered for adoption in the field. Then, the analyses in this item were carried out only considering specimens with 3% of AAC or more.

The stability of a dry stack of cemented material depends on two main characteristics. One is the compaction degree, and another is cement content. Following (Consoli, Da Silva, et al., 2018; Consoli et al., 2020), these parameters are referred to as porosity (η) and amount of binder (B_{iv}) and can be gathered into a unique parameter, the porosity/binder content index (η/B_{iv}), and expressed as follows (Eq. 1):

$$\frac{\eta}{B_{iv}} = \frac{100\cdot\left[1-\frac{\gamma_d}{\gamma_s\cdot(1+C/_{100})}+\frac{\gamma_d\cdot(C/_{100})}{(1+C/_{100})\cdot\gamma_c}\right]}{100\cdot\frac{(C/100)\cdot\gamma_d}{(1+C/_{100})}} \tag{1}$$

where γ_d is the dry unit weight of the specimen, γ_s is the unit weight of the solids of the iron ore tailings, γ_c is the unit weight of the solids of the alkali-activated cement, and C is the amount of alkali-activated cement expressed in percentage.

The porosity/binder content index has been used as an important geotechnical parameter in studies of soil improvement. So, verify its validity for tailings-AAC mixtures is essential, as is the case in this study.

4.1 UNCONFINED COMPRESSIVE STRENGTH

Unconfined compression tests have extensively been used for the definition of improvement level for several reasons, such as the accumulated experience with this kind of test, the low cost, simplicity, and speed. Moreover, the results for different binders and materials are available in the literature, providing a direct comparison of the materials tested (Cerveira et al., 2017; Consoli et al., 2007; Correia et al., 2019; Festugato et al., 2021). Figure 5 displays the unconfined compression (q_u) as a function of the η/B_{iv} index. For each curing period, it was possible to correlate q_u to the η/B_{iv} parameter through a power-type relationship, Eq. 2. The exponent was adopted as -1.30 because this was found as a usual value for sandy materials (Consoli et al., 2010, 2012, 2017; Diambra et al., 2017).

$$q_u(kPa) = A \times 10^4 \left(\eta/_{B_{iv}}\right)^{-1.30} \tag{2}$$

Likewise, Eq. 2 was adequate to model the strength results, as shown by the great R^2 values in Table 3. The decrement in the porosity implies the augment in the degree of interlocking between the particles, which favours the strength development (e.g. Consoli, Da Silva, et al., 2018; Consoli et al., 2007; Consoli, Winter, et al., 2018). Besides, it is evident that greater quantities of AAC (↑ B_{iv}) have led to strength gains owing to a more developed cement matrix (e.g. Cerveira et al., 2017; Consoli et al., 2021; Karatas et al., 2019). The cement phase evolves during the curing period, and the strength growth rate diminishes after 28 days.

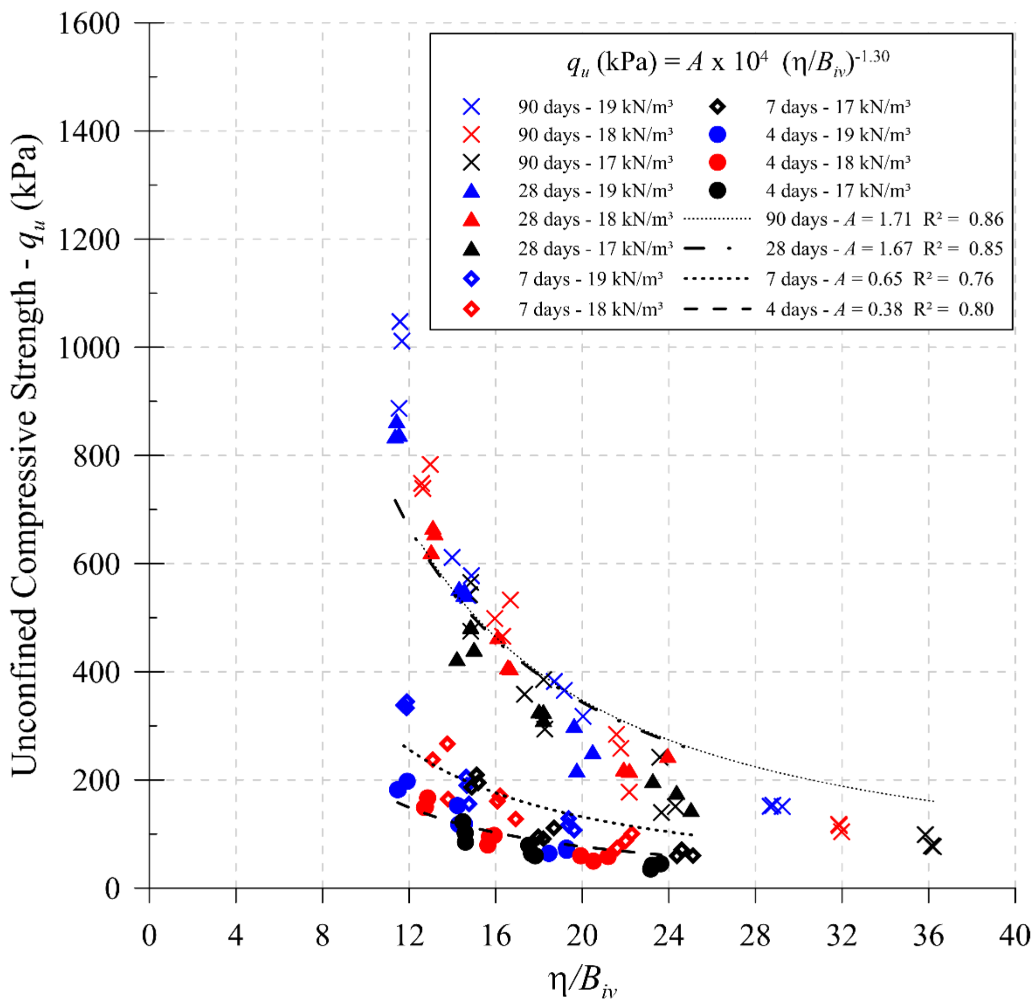


Figure 5: Unconfined compressive strength as a function of the porosity/binder content index

Table 3: Parameters for Eqs. (2) and (3)

Curing period	Stiffness data – G_0 (MPa)		Strength data – q_u (kPa)	
	A	**Coefficient of determination – *R²***	***A***	**Coefficient of determination – *R²***
90 days	1.66	0.96	1.71	0.86
28 days	1.44	0.98	1.67	0.85
7 days	0.69	0.84	0.65	0.76
4 days	0.36	0.79	0.38	0.80

The addition of the AAC to the tailings increased strength, especially for the case of 28 days of curing. This increased strength helps the fast development of the stacking, as higher strengths allow for the achievement of higher heights and steeper slopes. Furthermore, the bonding between particles proportioned by the cement suppresses the liquefaction potential in critical zones of the stack.

4.2 INITIAL SHEAR MODULUS

The measurement of shear wave velocities to determine G_0 is considered one of the most reliable methods (Iwasaki & Tatsuoka, 1977; Kramer, 1996). This modulus describes the behaviour of the soil under working loads and seismic movements and defines the dynamic properties of soils (Lee & Albaisa, 1974). Figure 6 exhibits the initial shear modulus results (G_0) outcomes as a function of the porosity/binder content index (η/B_{iv}). The approach for stiffness results was the same for strength results and yielded equivalent tendencies. Hence, a linear relationship between G_0 and the η/B_{iv} was obtained as follows (Eq. 3):

$$G_0(MPa) = A \times 10^4 \left({}^{\eta}/_{B_{iv}}\right)^{-1.30} \quad (3)$$

Table 3 summarizes the A scalar values and the respective coefficient of determination (R^2), considering each curing period for the compacted iron ore tailings-cement samples. A good agreement was obtained between the stiffness and the porosity/binder content index. Furthermore, the direct proportionality between G_0 and the η/B_{iv} ratio indicates that either (i) the increment in the compactness ($\downarrow\eta$) and the (ii) increase in the amount of utilized binder favour the stiffness development in an equivalent manner (Consoli et al., 2012, 2020; Corrêa et al., 2021). Both tendencies promote the increase in the contact area between tailings' particles and lead to G_0 gains (Yun & Santamarina, 2005).

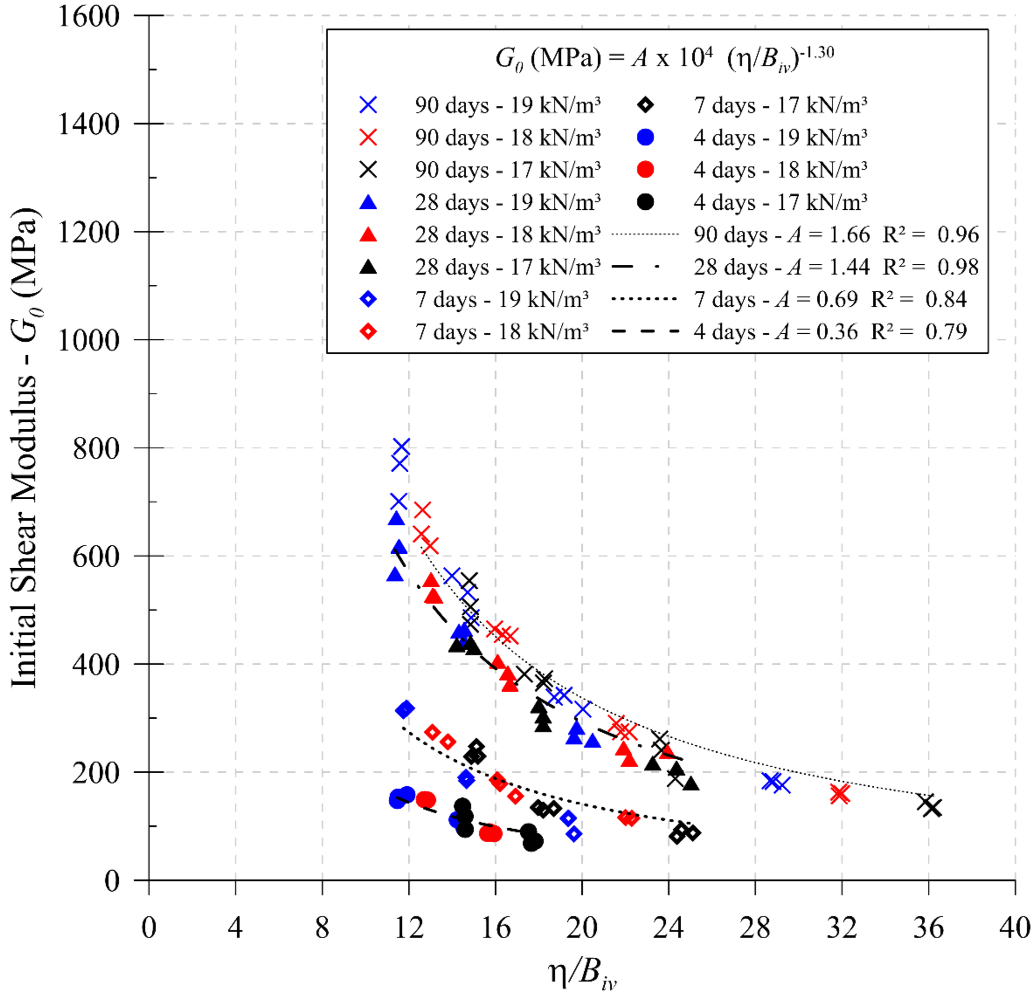


Figure 6: Initial shear modulus as a function of the porosity/binder content index

Following Diambra et al. (2017), the impact of the curing period is attested by the value of A, which has increased proportionally, whereas the other adjustment parameters remained constant. Moreover, the stiffness gain rate is much higher up to 28 days of curing, as indicated by the values of A depicted in Table 3. These results indicate that the development of the binding compounds is more intense during the early stages of curing than over the last. Thus, the material's stiffness tends to stabilize with time, associated with the timing-dependent development of cementitious reaction products (Chen, 2021; Kim & Kang, 2020; Pinheiro et al., 2020).

Compared with Consoli et al. (2022) the use of Portland Cement has induced higher stiffnesses and strengths than the ones achieved in this experimental program. However, a significant gain was achieved for a waste-based cement aligned with circular economy approach.

4.3 STATISTICAL ANALYSIS

An analysis of variance (ANOVA) was carried out to evaluate the statistical significance of the controllable factors (and their interactions) on the unconfined compressive strength (q_u) and initial shear modulus (G_0) of the samples. A level of significance (α) equal to 5% was adopted, and the data relative to the specimens moulded using 2% of the alkali-activated cement were neglected from the analysis. In this regard, Table 4 summarizes the ANOVA results for the strength tests, Table 5 exhibits the same for the stiffness tests, whereas Fig. 7 portrays the main effects plot for both.

Table 4: ANOVA table for the strength results

Source	Degrees of freedom	Sum of squares	Mean squares	*p* value
Model	35	6,228,712	177,963	0.000
Linear	7	5,309,124	758,446	0.000
CP	3	3,153,161	1,051,054	0.000
γ_d	2	534,443	267,221	0.000
C	2	1,621,521	810,760	0.000
Interactions	16	859,204	53,700	0.000
CP*γ_d	6	180,925	30,154	0.000
CP*C	6	559,574	93,262	0.000
γ_d*C	4	118,705	29,676	0.000
Interaction	12	60,383	5,032	0.000
CP*γ_d*C	12	60,383	5,032	0.000
Error	72	52,690	732	-
Total	107	6,281,402		

Table 5: ANOVA table for the stiffness outcomes

Source	Degrees of freedom	Sum of squares	Mean squares	*p* value
Model	17	3,103,451	182,556	0.000
Linear	7	2,979,658	425,665	0.000
CP	3	2,262,673	754,224	0.000
γ_d	2	124,225	62,112	0.000
C	2	996,386	498,193	0.000
Interactions	10	75,386	7,539	0.000
CP*γ_d	6	61,063	10,177	0.000
γ_d*C	4	24,918	6,230	0.015
Error	81	153,200	1,891	-
Total	98	3,256,651		

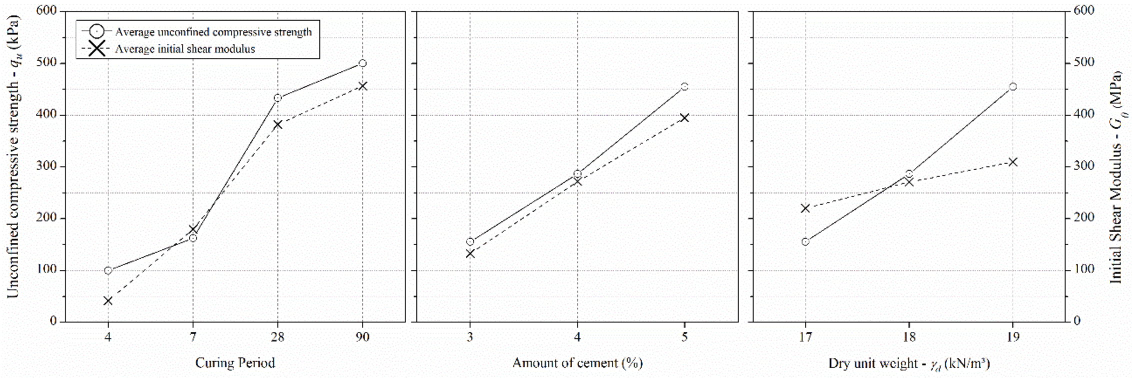


Figure 7: Main effects plot for the strength and stiffness results

All the main factors and interactions are statistically significant in altering the response variables (q_u and G_0) at the adopted level of significance (i.e., p values < 5%). This output means that the strength and stiffness of the cemented IOT were sensitive to alterations in the controllable variables (i.e., curing period, dry unit weight, and amount of cement). For example, the changing caused in q_u and G_0 by the rise on the dry unit weight, from 17 kN/m³ to 18 kN/m³, were statistically significant. The influence of the single factors (strength and stiffness) is more significant than the effect of their interactions, as demonstrated by the magnitude of the sum of squares values.

The main effects plot (Fig. 7) demonstrates that the q_u and G_0 results were similarly affected by the controllable variables, corroborating the trends previously described using the η/B_{iv} based relationships. It is visible the more significant influence of the curing period (CP) over the dry unit weight (γ_d) and amount of cement (C). The role of the dry unit weight is related to the degree of interlocking between the particles, whereas higher amounts of cement favour the binding between the tailings' constituents (e.g. Consoli, Da Silva, et al., 2018; Consoli, Winter, et al., 2018; Corrêa et al., 2021). In addition, a more compacted structure benefits the development of the cement matrix, which explains the statistical significance of the γ_d - C interaction. The formation of the binding compounds occurs over time which attests to the great effect exerted by the curing period and, as well, the significance of the interaction amidst the curing period and amount of cement (e.g. Correia et al., 2019; Pinheiro et al., 2020).

4.4 LIFE CYCLE ASSESSMENT

Figure 8 and Table 6 shows the results of the life cycle assessment established in this work. For the eleven impact categories evaluated, the binder that uses a production technique with mining tailings allowed a reduction in the environmental impact in 10 categories in the stabilization of 100 m³ of tailings for stockpiling. On average the binder provided a reduction of 29% compared to the conventional binder.

For the two binders compared, the production of sodium silicate generates a greater contribution in the impact categories and, consequently, metakaolin generates a lower impact even though it represents the largest portion of material in the binder. The production of sodium silicate uses sodium hydroxide in its production. According to Hong et al. (2014), the production process of sodium hydroxide involves the use of significant quantities of electricity, raw salt, sulfuric acid, sodium carbonate, sodium sulphite, and other materials. As a result, sodium hydroxide has a greater impact on the environment in terms of its contribution to environmental factors.

The reduction in energy for the production of sodium silicate, with the technique proposed by Vogt & Lameiras (2019), directly saved the consumption of 17.58 x10³ MJ. The use of mining tailings for the production of SS and MK was possible to save 7.42 tons of natural resources in the production of AAC, also in a direct way. The main categories of impacts affected were cumulative energy demand, global warming, terrestrial ecotoxicity, photochemical oxidation, acidification and eutrophication. Therefore, the resulting change in AAC production has made it possible to create an environmentally friendly production chain and a more sustainable tailings stabilization. Along with the reuse of tailings and reduction of waste, the new binder production chain performance demonstrates that it contributes towards CE.

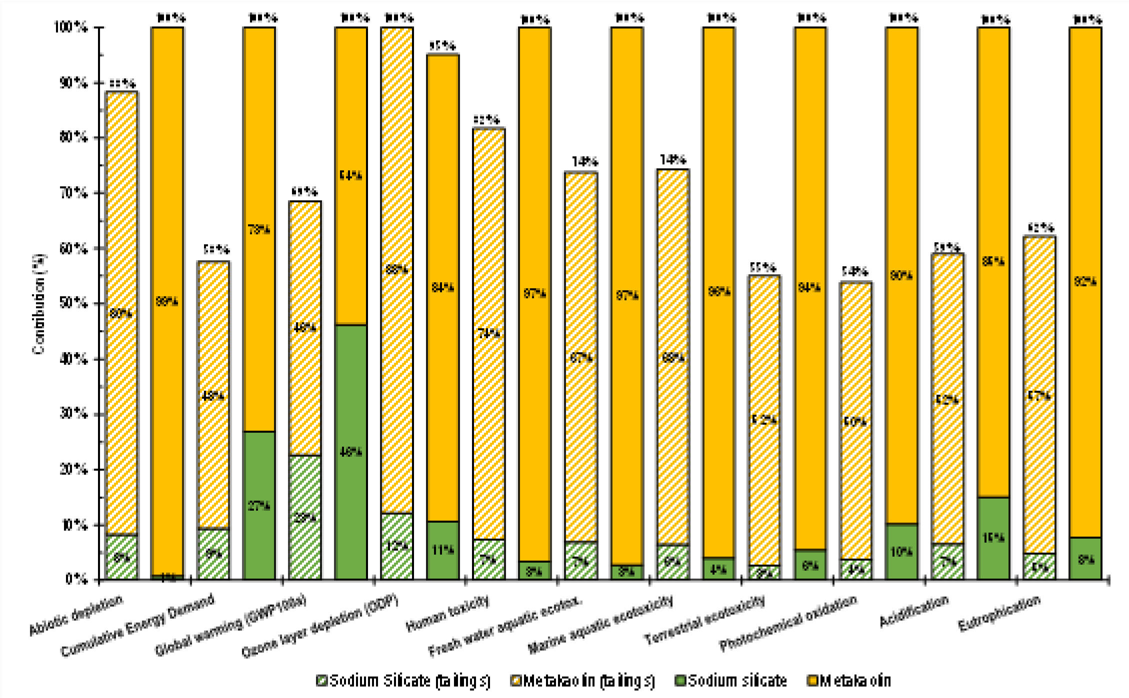


Figure 8: Eco-profile of AAC with tailings compared to conventional AAC

Table 6: Environmental impacts for the stabilization of 100m³ of iron ore tailings

Impact Category	Unity	Alternative AAC	Conventional AAC
Abiotic depletion	kg Sb_{eq}	$2.60x10^{-2}$	$2.94x10^{-2}$
Cumulative Energy Demand	MJ	$4.53x10^{4}$	$7.87x10^{4}$
Global warming	kg CO_{2eq}	$2.30x10^{3}$	$3.35x10^{3}$
Ozone layer depletion	kg CFC-11_{eq}	$1.27x10^{-3}$	$1.20x10^{-3}$
Human toxicity	kg 1,4-DB_{eq}	$1.39x10^{3}$	$1.70x10^{3}$
Fresh water aquatic ecotox.	kg 1,4-DB_{eq}	$8.76x10^{2}$	$1.19x10^{3}$
Marine aquatic ecotoxicity	kg 1,4-DB_{eq}	$1.51x10^{6}$	$2.03x10^{6}$
Terrestrial ecotoxicity	kg 1,4-DB_{eq}	4.97	9.04
Photochemical oxidation	kg C_2H_{4eq}	0.57	1.06
Acidification	kg SO_{2eq}	8.65	14.7
Eutrophication	kg PO_{4eq}	2.16	3.47

5 CONCLUSIONS

The present research evaluated the technical and environmental viability of a novel alkaline-activated cement produced from mining waste. Using this new binder could enhance several aspects of the mining operation and cope with adopting circular economy in mining. Adapting the ore extraction industry to environmental aspects is a long process and involves the whole operation. However, the valorisation and reduction of waste generated in extraction and correct destination are some points to be improved. The methodology herein presented uses a waste-based source of silica and an alkaline-activator for use in dry stacks, relating all these issues in one application. Furthermore, the small amounts of binder introduced could increase the structure's performance while still allowing the reprocessing of the tailings in the future. Thus, considering the boundaries of the study, specific conclusions can be drawn:

The porosity/binder content index could express the stiffness at small strains and the strength of the compacted cemented tailings for all the tested curing periods. The obtained curves can be used as dosage curves to predict of a required mechanical response for stacking filtered dry tailings.

Statistically, the curing period was the most important factor in modifying the strength and stiffness response of the mixtures studied. Increases in strength and stiffness greater than 2.5 times were reported between 7 and 28 days of curing.

The increased strength due to AAC addition allows achieving steeper slopes and higher heights in the dry stacking deposition method, which leads to a decrease in the new area impacted by the mining activity.

The AAC considered, which is derived from the waste and tailings, reduces in waste generation in mining operation and adds value to it composing a new binder that can be used for several activities.

The use of AAC with tailings presents an environmentally sustainable option for stabilizing iron tailings when compared to traditional alkaline binders. The energy and natural resources saved in their production provided superior performance in 10 of 11 environmental impact categories assessed.

NOTATIONS

B_{iv}	volumetric binder content
η	porosity
γ_d	dry unit weight
γ_s	unit weight of solids of the iron ore tailings
γ_c	unit weight of solids of the alkali-activated cement
w	moisture content
q_u	unconfined compressive strength
G_0	initial shear modulus
ρ	bulk density
ANOVA	analysis of variance
AAC	alkali-activated cement
OPC	ordinary Portland cement
IOT	iron ore tailings
MK	metakaolin
SS	sodium silicate
C	amount of alkali-activated cement expressed in percentage
UPV	ultrasonic pulse velocity
R^2	coefficient of determination
V_s	shear wave velocity

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INNOVATIVE USE OF RECYCLED RUBBER AND MINING BY-PRODUCTS FOR SUSTAINABLE RAIL AND ROAD INFRASTRUCTURE

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ABSTRACT

Encouraging more real-life applications of circular economy perspectives in transportation infrastructure design and construction, this paper focuses on utilising granular wastes (i.e. coal wash and steel slag) from coal and steel mining for port reclamation, and recycled rubber elements including granulated rubber particles, rubber mats, tyre cells and truck tyre segments for stabilising track formations and reducing ballast degradation. The mixtures of coal wash and steel slag were optimised through a proposed novel customer-made selection criteria and verified through field trial. Moreover, the promising damping property of rubber (with respect to strain energy capacity) was fully exploited to design substructure energy retention layers to minimise deformation and degradation of track elements including impact damage caused by track irregularities such as rail corrugations. The large-scale laboratory testing results obtained using the large-scale triaxial, Process Simulation Prismoidal Triaxial Apparatus, and the prototype National Facility for Cyclic Testing of High-speed Rail and the field trial verify that rail tracks altered with the above-mentioned rubber elements easily satisfy the specified standards and are even superior to conventional ballast tracks in terms of degradation, deformation, stress distribution, and track vibration. In addition, these environmentally friendly approaches promote the reuse of mining by-products and discarded tyres and conveyor belts in transportation infrastructure while providing long-term cost benefits that can save millions of dollars annually in track maintenance and quarrying natural rock aggregates.

1 INTRODUCTION

According to the Australian Infrastructure and Transport Statistics (2022), domestic freight transport in Australia has been experiencing significant growth over the past 40 years, with road, rail freight and coastal shipping activities dominating the sector. It is projected that the demand will continue to increase exponentially. Rail and road maintenance costs have been increasing due to the growing volume and intensity of freight transportation on existing infrastructure. The incorporation of waste and marginal materials in new rail and road construction projects as well as port reclamation projects, not only extends infrastructure longevity but also offers a sustainable approach to address the growing accumulation of waste materials, such as used tyres, industry by-products like coal wash (CW), and Basic Oxygen Steel furnace slag (BOS).

CW and BOS are two common granular by-products from coal mining and steel production, respectively. In Australia, millions of tons of these waste materials are generated every year (Malasavage et al., 2012, Indraratna et al., 2018). Rather than dumping them on useable lands, paying government levies, and causing environmental issues, both industries and the government expect novel solutions to reuse these materials in civil and geotechnical engineering. CW, albeit with a high particle breakage potential, has been proven to be a promising water-front embankment fill once compacted well (Rujikiatkamjorn et al., 2013). The volumetric instability (expansion) of BOS hinders its individual usage in civil/geotechnical engineering, but well-balanced mixtures by incorporating other materials such as CW, rubber crumbs, cement, asphalt, etc. have been investigated and verified their broad usage in roads, rail tracks, port reclamation and civil constructions (Wang, 2010, Malasavage et al., 2012, Indraratna et al., 2020, Qi and Indraratna, 2022a). For instance, a blended mixture of CW and BOS, has been tested and applied in a real-life port expansion project located on the south coast of New South Wales (Port Kembla Reclamation project). This mixture has been proven to meet the expected requirements for bearing capacity, drainage, swelling potential, compaction, and shear strength properties through laboratory and field investigations and testing facilities (Chiaro et al., 2015).

Over 50 million waste rubber tyres are produced in Australia each year, posing a severe problem due to excessive dumping and uneconomical recycling techniques that result in massive stockpiles (Mountjoy et al., 2015). Additional challenges arise while handling enormously huge and heavy off-the-road (OTR) tyres produced by the mining industry. Due to the high energy-absorbing capacity and damping properties of recycled rubber materials, they have been recently brought to seismic-isolation projects and transportation infrastructure such as railways, roads, and airport runways (Tiwari et al., 2012, M Sol-Sánchez et al., 2015, Arachchige et al., 2022, Qi and Indraratna, 2022b). Given the need for faster and heavier rail lines, an innovative solution to reuse these tyres can provide promising outcomes to enhance railway substructures.

The typical products of recycled rubber used in rail tracks are granulated rubber (or rubber crumbs), rubber mats (e.g. under ballast mats, under sleeper pads), tyre cells, rubber geogrids, and arch-shaped rubber tyre segments. The granulation of used rubber tyres is a widely adopted practice, with tyre-derived aggregates commonly employed for energy recovery, rubberised asphalt, and concrete applications (Navarro and Gámez, 2012, Mohajerani et al., 2020). Moreover, recent research has highlighted the feasibility of incorporating rubber granules into conventional ballast for rail infrastructure. Rubber intermixed ballast system is an experimentally proven approach wherein the certain grain size of conventional ballast is partially substituted with rubber granules through the optimisation of the geotechnical characteristics of the ballast-rubber mixtures (Arachchige et al., 2021). Recent studies (e.g. Sinniah K Navaratnarajah and Indraratna, 2017, Ngo et al., 2019) found that the reuse of rubber sheets is worth considering in terms of track performance, sustainability, and economic perspective. For instance, rubber mats are placed under the ballast layer (under ballast mats; USB) to mitigate the degradation of ballasted railway tracks while reducing energy transferred into the subsequent track formation, including the subgrade. In addition, trial tracks were implemented as a real-life application, where waste tyre segments infilled with granular wastes served as the capping layer for railways (Indraratna et al., 2022b). These trials followed comprehensive experimental investigations, which involved large-scale laboratory physical simulations and testing using the process simulation prismoidal triaxial apparatus (PSPTA) and the National Facility for Cyclic Testing of High-speed Rail.

This paper describes four distinctly different innovative and cost-effective approaches including: (i) port reclamation using the optimal mixtures of CW and BOS, (ii) an alternative load-bearing granular mass, namely, Rubber Intermixed Ballast System (RIBS), developed by replacing a fraction of rock aggregates in the ballast matrix with similar size rubber granules (10% by weight), (iii) ballast mats (UBMs) installed on top of the concrete deck to reduce ballast deformation and degradation, and (iv) a hybrid track built using tyre cell assembly in tandem with giant off-the-road (OTR) arch-shaped tyre segments installed along the track shoulders to prevent the lateral movement of ballast, thus minimising ballast dilation and breakage. The optimal blends of CW and BOS from approach (i) were established to be used as structural fill based on comprehensive laboratory test results and then verified through a field trial for port reclamation. The novel rubber inclusions from approaches (ii)-(iv) aimed to provide an equivalent energy reservoir to reduce the dissipated energy from moving rollingstock to other track substructure elements such as ballast and concrete sleepers, thus minimising deformation and damage. Large-scale laboratory programs using the large-scale triaxial apparatus, the iconic Transport Process Simulation Triaxial Facility, and Australia's first and only National Facility for Cyclic Testing of High-speed Rail were conducted to verify these concepts. Recently, they were adopted as part of the track construction by Sydney Trains.

2 USE OF WASTE GRANULAR MIXTURES IN PORT RECLAMATION

2.1 MATERIALS

Port Kembla has been Wollongong's commercial harbour solely catering to the coal and steel export market. The coal mining and steel industry generates a massive volume of coal wash, and blast furnace slag up to 2.1 million tonnes per year. To attract new trades and increase the volume of existing berths, a new 45-hectare outer area was constructed via land reclamation. Due to the associated environmental concerns and the scarcity of natural fill, the Port Kembla Port Corporation considered using these locally abundant waste aggregates as the predominant reclamation fill. In-depth knowledge of the behaviour of heterogeneous waste materials through their geotechnical characterisation was acquired on the basis of load-bearing capacity and post-construction deformation. Based on the trace element concentration tests conducted by the Port Kembla Port Corporation (PKPC), both coal wash and furnace slag pose little or no contamination risk to the environment or public health (Coffey, 2009). Although numerous studies since the 1990's have reported on the geotechnical characterisation of heterogeneous coal wash, minimal information is still available to describe comprehensively their short and long-term behaviour on compaction due to particle breakage. Indraratna et al. (1994)

showed that the geotechnical properties of coal wash can be markedly improved with appropriate compaction. Applying steel slag alone can pose a severe problem of swelling (Chiaro et al., 2015). The optimum mixture of both materials can minimise the adverse effects and provide a suitable blend to be used as fill.

For compacted fill, shear strength and permeability are the main criteria where a friction angle greater than 30° or CBR of at least 10% (Davies and McIlquham, 2011) and the permeability within the range between 1×10^{-6} and 1×10^{-4} cm/sec (Look, 2007). The purposes of the criteria are to control post-compaction settlement and prevent the generation of excess pore water pressure and internal erosion. Under relatively low overburden pressure, the free swelling should be within 3%, whereas the breakage index of the waste mixture should be within 12%. Chiaro et al. (2015) showed that the optimum blend should contain 35-60% coal wash by weight (Figure 1).

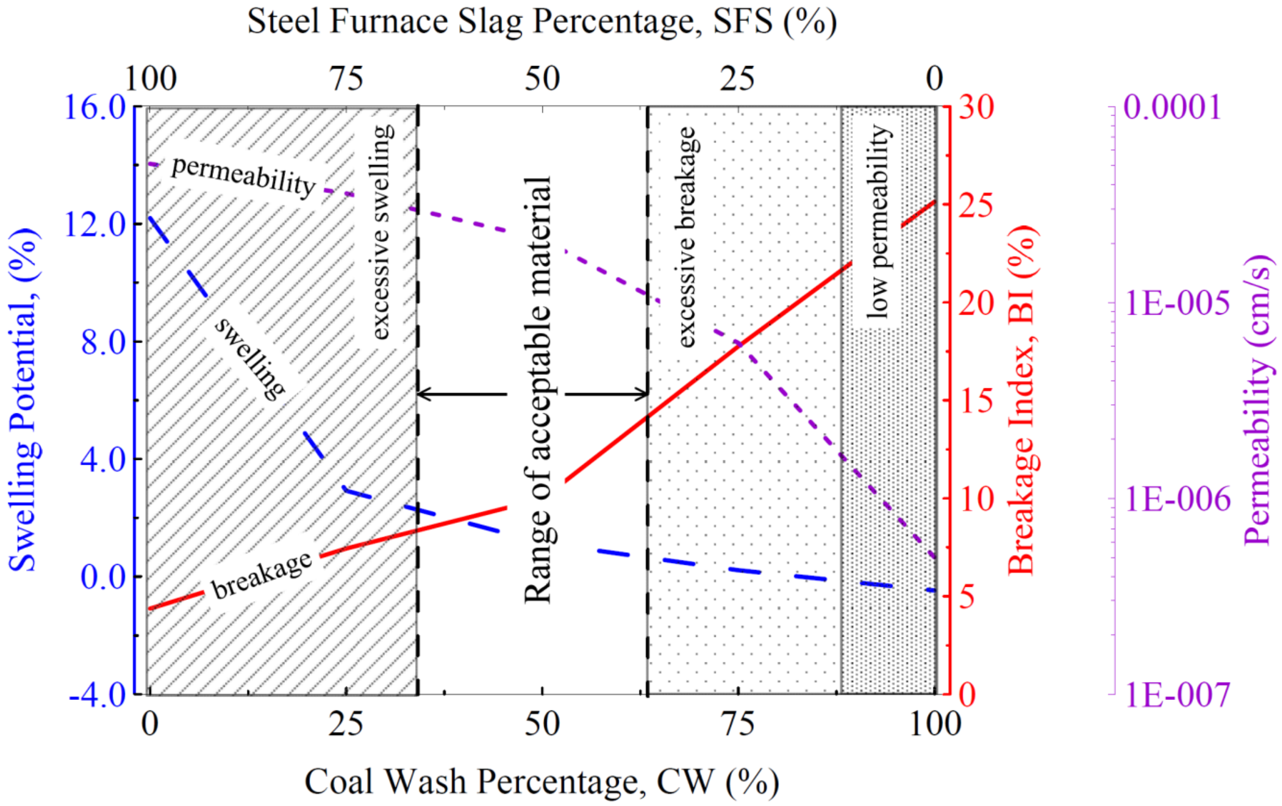

Figure 1: Determination of CW and BOS blend for port reclamation (modified after Chiaro et al., 2015)

2.2 FIELD TRIAL

The field verification was conducted at the Port Kembla Outer Harbor reclamation site with an area of $55 \times 14 m^2$ with 1.4 m below the ground level. The area was separated into two subsections for two mixtures (43% CW and 27% CW mixtures). The blending of the materials was carried out by an excavator and the materials with an initial layer thickness of 0.3 m were compacted using a 13-tonne smooth steel drum roller, as shown in Figure 2. The field density was determined using sand cone replacement and nuclear densitometer tests. The laboratory maximum dry unit weights for 27% and 43% CW are 21.1 and 20.2 kN/m^3, respectively. Four roller passes provide sufficient energy to achieve a 90% relative compaction. Based on particle size distribution analyses, most of the large particles (>2cm) were degraded to smaller particles, and the overall gradation was similar to that of laboratory specimens.

To assess the shear strength of the mixtures with the depth, dynamic cone penetration tests (DCPTs) were conducted at 0.1 m intervals. The equivalent California bearing ratios are in the range of 25-50, similar to that of dense to very dense sand (Figure 3), confirming its suitability as structural fill in terms of shear strength.

Kindly take note that the outcomes of this investigation were constrained to a specific coal wash and steel furnace slag within the Illawarra region. Given the substantial alterations in the characteristics of these materials, there exists a valuable opportunity to delve into the examination of fluctuations in geotechnical properties. This examination would primarily centre around the impact of free lime content and the calibre of fines present in the coal wash.

(a) (b)

Figure 2: Compaction trial (a) Coal wash and steel furnace slag and (b) Vibration roller

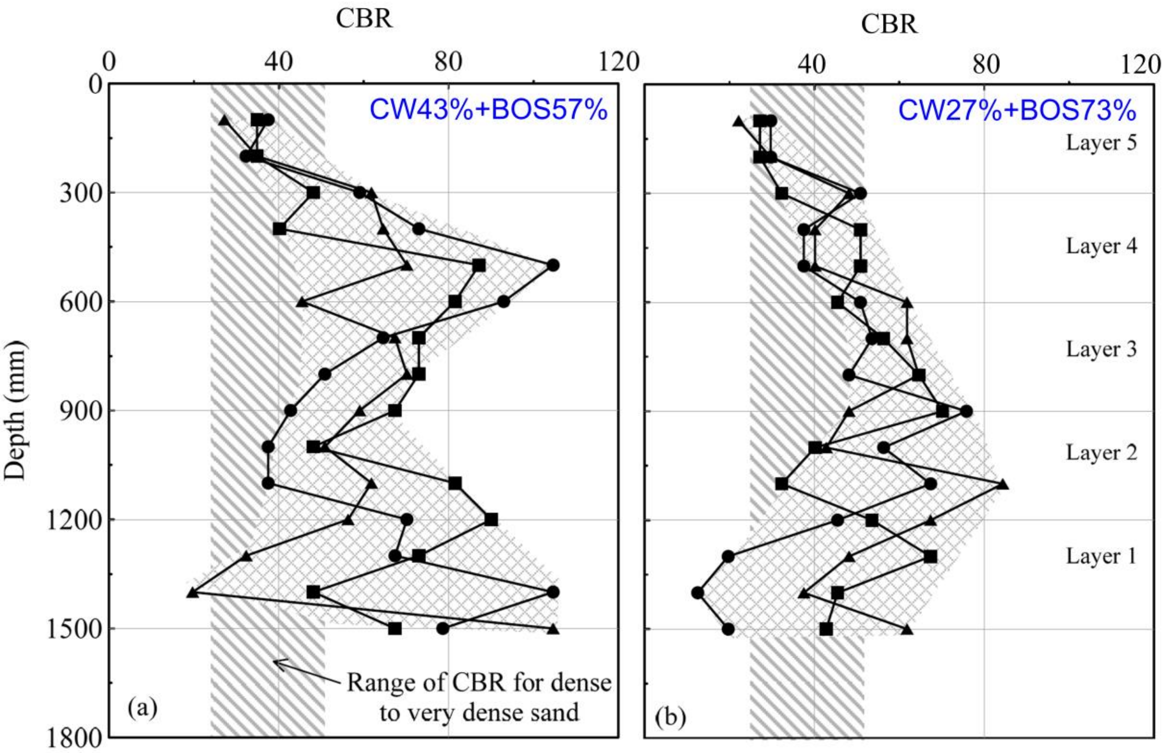

Figure 3: Equivalent CBR changing with depth for (a) 43% coal wash mixture and (b) 27% coal wash mixture (Tasalloti et al., 2015)

3 USE OF RECYCLED RUBBER GRANULATES WITH BALLAST IN RAIL CONSTRUCTION

Owing to restrictions in international shipping, transportation and storage limitations, and the lack of substantial domestic demand for products made from waste tyres, the Australian government's Department of Climate Change, Energy, the Environment, and Water included End of Life Tyres in the Minister's priority list for 2022-2023 (2022), marking the first instance in history. The utilisation of recycled rubber granulates in rail construction is one of the promising applications that can be developed as industry-led projects to elevate domestic consumption and enhance the onshore recovery rate of end-of-life tyres. Recently, a comprehensive study which involved large-scale laboratory simulations and field trials, was conducted to evaluate the potential applicability of rubber-mixed ballast as an alternative granular medium to the conventional ballast layer. The study used rubber granulates derived from discarded tyres, with sizes ranging from 9.5

mm to 19 mm (Arachchige et al., 2021, 2022). These granulates were mixed in varying proportions, from 0% to 15%, while ensuring compliance with Australian ballast standards (AS:2758.7, 2015) in the final mixture, the Rubber Intermixed Ballast System (RIBS).

3.1 PROPERTIES OF RUBBER INTERMIXED BALLAST SYSTEM

Shearing resistance is crucial in ballast, as the ballast layer in the track substructure serves as a load-bearing layer subjected to dynamic moving loads. At the same time, the dilation angle characterises the tendency of granular materials to dilate under applied loads. Figure 4*a* shows the effective friction angles (φ_{ef}), the mobilised friction angles (φ_p), and dilation angles (ψ_p) at peak stress ratio (η_{peak}) of RIBS mixtures with varying rubber percentages obtained from the large-scale consolidated drained triaxial tests. As the increased rubber content (R_b%) reduces the dilation, increasing effective confining pressures (σ_3') further contribute to the reduction in dilation, owing to a well-compressed particle arrangement. The φ_p at the peak stress ratio encapsulates the impact of both breakage and dilatancy within the sample; consequently, φ_p decreases with increased R_b% due to the reduction in breakage and dilation.

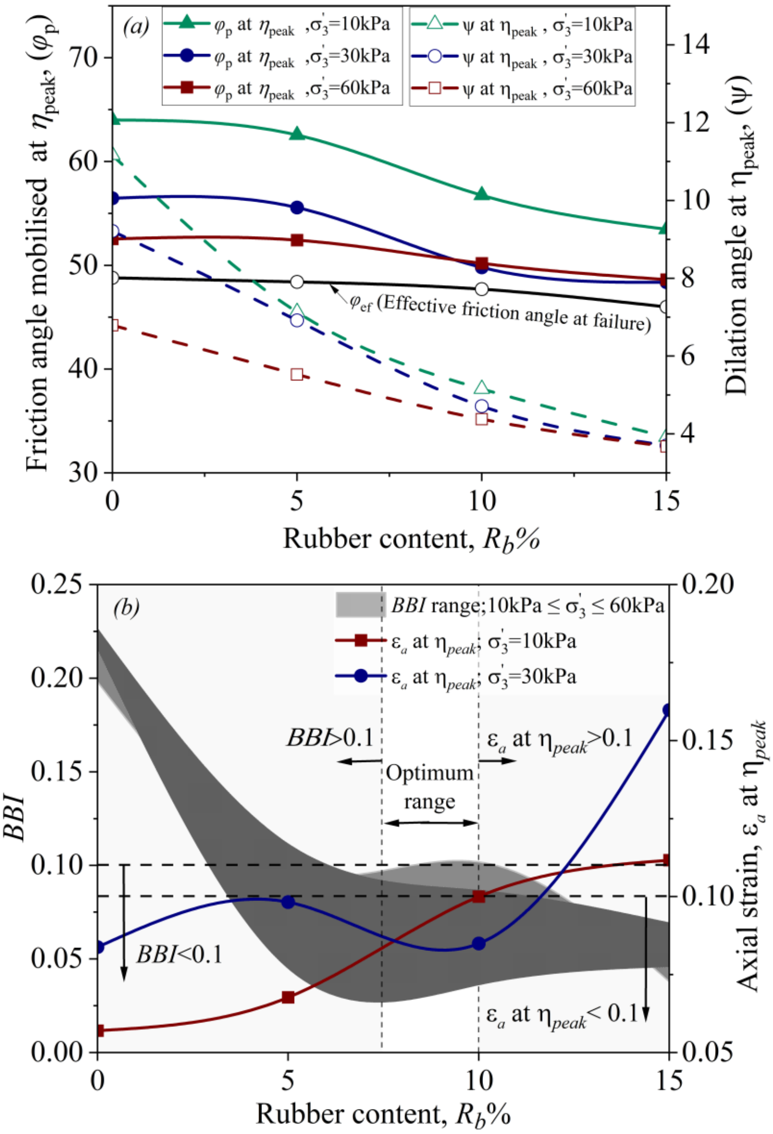

Figure 4: (*a*) Effect of the rubber on friction angle and dilation angle at the peak stress ratio (b) Effect of the rubber on ballast breakage and axial strain (modified after Arachchige et al., 2021)

The decrease in particle breakage with increased $R_b\%$ is evident in Figure 4b. Note that an increase in $R_b\%$ from 0 to 15% leads to a minor change (< 6% reduction) in the effective friction angle for RIBS mixtures from 48.8° to 46.0°. The breakage of ballast during the tests is assessed by using the Ballast breakage Index (BBI) following the method outlined by Indraratna et al. (2005). As presented in Figure 4b, even a minor addition (5%) of rubber significantly reduces particle breakage, owing to the cushioning effect and the soft particle interaction within the mixture. Nevertheless, an increase in $R_b\%$ beyond 5 -10% does not yield a substantial improvement in controlling particle breakage. Moreover, according to Indraratna et al. (2015), the axial strain (ε_a) of conventional ballast remains below 0.1 under typical track conditions during laboratory testing conducted with large-scale apparatus. Hence, one of the other key factors considered when determining the optimal rubber content in RIBS is the maximum allowable axial strain of 0.1. As illustrated in Figure 4b, the introduction of rubber into the mixture leads to an increase in ε_a with a substantial rise observed when $R_b\%$ exceeds 10%. Based on the above, the optimal $R_b\%$ added to RIBS is determined to be 10% by weight.

3.2 FIELD TESTS ON RIBS AT CHULLORA

Following a thorough analysis of laboratory test results, RIBS with an optimal rubber content of 10% was implemented in a 20-m instrumented trial track at Chullora, near Sydney. The conventional ballasted track construction methods could be adapted for RIBS tracks, except for placing RIBS from reduced heights (< 1m). Figure 5 (*a*-b) illustrates the onsite blended RIBS and placement of RIBS in a 20 m stretch atop the geotextile positioned on the capping layer. This adjustment was made to manage the segregation of rubber particles caused by their lower specific gravity in comparison to natural rock aggregates. The RIBS material was laid on top of the prepared capping layer, effectively substituting a 150 mm bottom ballast layer in conventional ballasted tracks. To mitigate vertical deformations over the service period, RIBS tracks were densely compacted using an augmented number of roller passes. It is noteworthy that visual observations indicated a comparably minimal particle breakage under roller compaction of RIBS, as opposed to the ballast breakage observed during the compaction of conventional ballast. Instrumentation was employed on both the RIBS track and the conventional track stretch to gather data for a comparative assessment of performance and lifespan. The instruments have been configured to capture data essential for analysing key track parameters, including vertical pressure distribution, settlement, ballast dilation, ballast breakage, vibration, and noise.

(a) (b)

Figure 5: (*a*) Onsite blended RIBS (b) Placement of RIBS on track at reduced height (1 < m)

4 USE OF WASTE RUBBER SHEETS FOR ENHANCED TRACK PERFORMANCE

Incorporating artificial inclusions like rubber sheets into rail track foundations absorbs energy and improves the soil-structure interaction, while reducing particle deterioration, offering an economically viable approach to enhancing track efficiency and prolonging maintenance intervals. The implementation of these rubber sheets within rail tracks can effectively dampen the impacts of dynamic stresses caused by moving wheels, thereby curbing track wear. These flexible components are typically made from materials such as rubber, high-density polyethene (HDPE), polyurethane elastomers (PU), thermoplastic polyester elastomer (TPE), and ethylene vinyl acetate (EVA) (Kaewunruen and Remennikov, 2006, SK Navaratnarajah and Indraratna, 2020). In recent times, elastic components have been produced by recycling discarded materials like worn-out tires, with the aim of stabilising the ground for constructing various infrastructures (Tiwari et al., 2012, Miguel Sol-Sánchez et al., 2014, Sinniah K Navaratnarajah and Indraratna, 2017). A track composed of concrete sleepers placed on a ballasted bridge deck demonstrated elevated rigidity and insufficient track damping, resulting in pronounced dynamic stresses within the substructure. Hence, careful attention to track damping is essential when exploring strategies to alleviate dynamic stresses and adverse vibrations (Sasaoka, 2006).

Enhancing the damping characteristics of the track structure involves incorporating a shock-absorbing pad into the ballasted track structure at an appropriate location. While elastic components closer to the track surface (such as rail pads and under sleeper pads) contribute to increased track elasticity, Under Ballast Mats (UBMs) prove more effective in mitigating rapid ballast deterioration in situations involving inflexible substructures like bridges and tunnels. Notably, areas with reduced ballast thickness or the substitution of lightweight timber sleepers with heavier concrete sleepers experience amplified stresses and accelerated deterioration in ballasted tracks. Implementing UBMs in these specific areas counteracts excessive degradation of ballast and counterbalances the adverse impacts linked to shallow ballast depths (Potocan, 2010).

4.1 LARGE-SCALE LABORATORY TESTING

A laboratory simulation was conducted by mimicking a track on a concrete bridge deck using the large-scale Process Simulation Prismoidal Triaxial Apparatus (PSPTA) to study the effect of a ballasted track stabilised with UBM made from discarded tyres. This facilitated the study of the performance of UBM when placed atop a concrete foundation. In this study, 10 mm in thick dense rubber mats (with a static stiffness of 0.2 N/mm³) were employed on the surface of the concrete. The laboratory tests involved using two different axle loads (25t and 35t) and frequency of cyclic loading ranging from 10 to 25 Hz. The process included various steps as depicted in photographs, such as setting up the PSPTA cubical triaxial chamber (Figure 6a), creating the concrete base (Figure 6b), placing UBM on the concrete base (Figure 6c), and the final test sample (Figure 6d). The study collected information on both vertical and lateral deformations, as well as the degradation of ballast, under conditions with and without placing UBM on the surface of the concrete foundation. Additionally, an empirical model for energy dissipation characteristics of ballast was proposed to predict the performance of the ballast layer integrated using UBM.

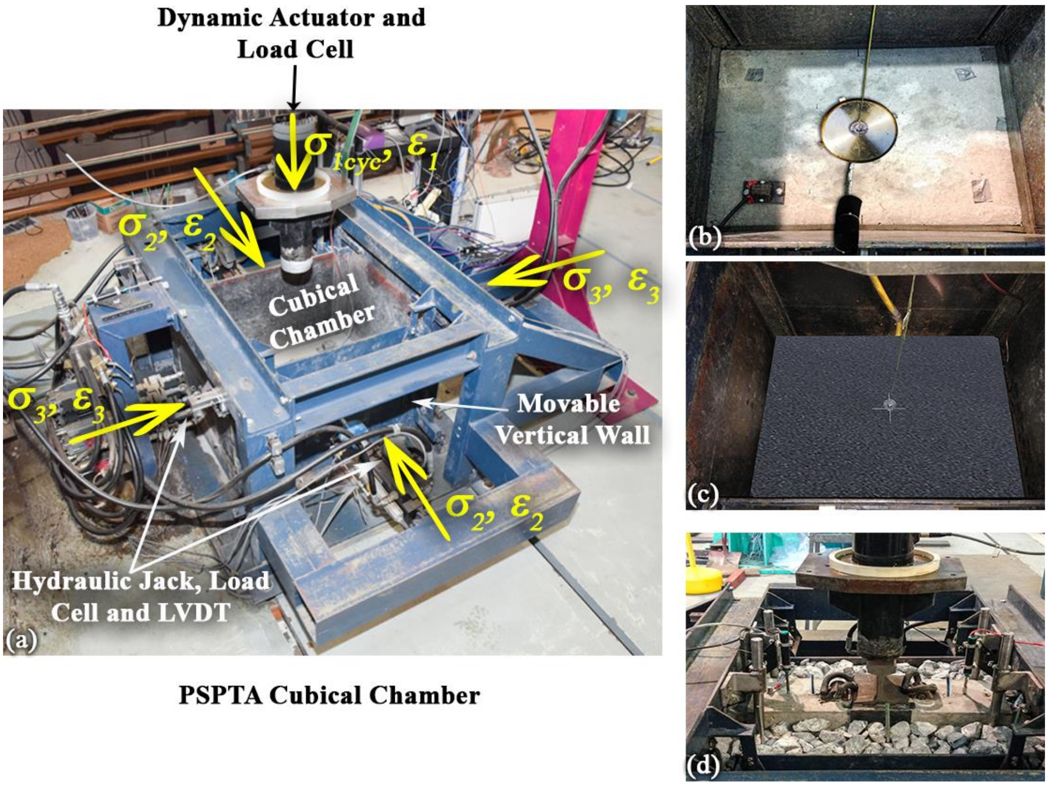

Figure 6: (a) PSPTA cubical triaxial chamber; (b) concrete base; (c) UBM on top of the concrete base and (c) final test sample (Modified after Navaratnarajah and Indraratna, 2017)

4.2 DEFORMATION AND DEGRADATION

Figures 7a and 7b illustrate the cumulative plastic deformations in the vertical and lateral directions, both with and without UBM, for axle loads of 25t and 35t. In each load, under various frequencies and UBM conditions, the plastic deformation of the ballast exhibited swift progression until around 10,000 cycles. Afterwards, the rate of settlement gradually decreased, maintaining stability with a nearly constant settlement as the loading reached around 100,000 load cycles. This initial rapid plastic deformation emerged from the rearrangements of differently sized ballast aggregates within the ballast mass, coupled with the abrasion of the surface and attrition of sharp-angular fresh ballast particles. Throughout this rapid deformation phase, the ballast particles compacted further, progressively reducing the potential for particle rearrangement until reaching a stable state.

Subsequent minimal plastic deformation was attributed to the continued degradation of well-contained particles due to repeated loading. To evaluate particle degradation, the study employed the BBI as recommended for ballast materials by Indraratna et al (2005). The quantified BBI are depicted in Figure 7c for axle loads of 25t and 35t under cyclic loading frequencies ranging from 10 to 25 Hz for ballast with/without UBM atop the solid concrete base.

The outcomes reveal that the application of UBM atop a rigid base can effectively diminish the overall plastic deformation of the ballast. This study also indicates the potential for a reduction of around a 10-20% decrease in vertical plastic deformation and roughly a 5-10% decrease in lateral plastic deformation. As anticipated, ballast degradation escalates with higher loads and frequencies of cyclic loading. Notably, the introduction of UBM yields a significant decline in ballast degradation atop the robust concrete base. The present investigation demonstrates an average reduction of 35-45% in breakage, highlighting the positive impact of UBM integration. Additionally, UBMs utilized in this research were produced locally using recycling discarded tyre waste. From a national perspective, crafting these rubber sheets from recycled rubber tyres would not only decrease the accumulation of tyre waste at disposal sites but also enhance land utilization, particularly in high-cost urban regions. Additionally, the resultant savings resulting from reduced demand for new aggregates through quarrying would present a direct environmental advantage, mitigating undue disruption of landscapes.

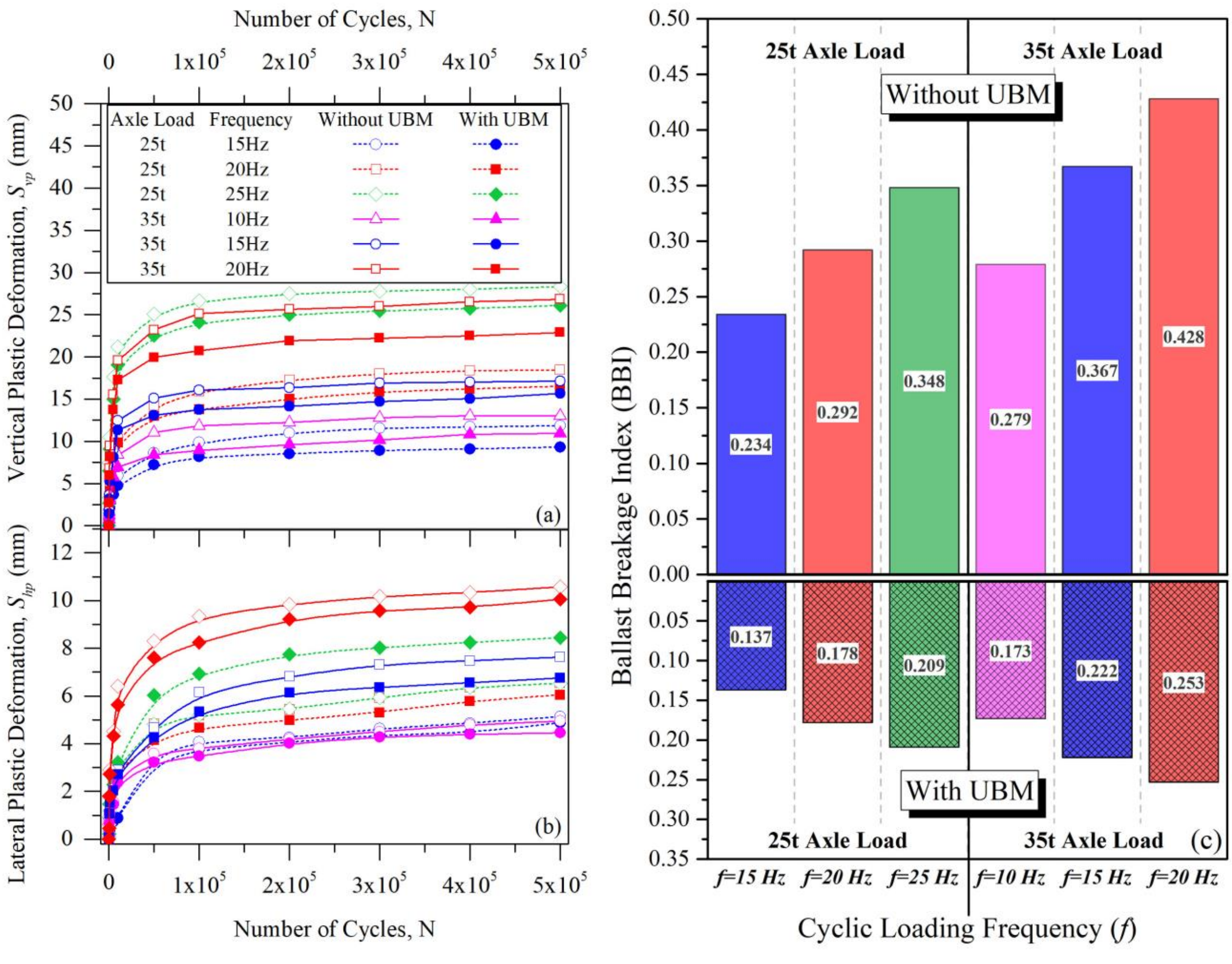

Figure 7: Ballast plastic deformation (a) vertical, (b) lateral; and (c) Ballast Breakage Index (Sinniah K Navaratnarajah and Indraratna, 2017)

4.3 ENERGY DISSIPATION

In this investigation, an empirical equation has been developed to quantify the overall dissipation of energy (E_D) per unit volume of ballast. This is achieved by incorporating separate dissipation components associated with plastic sliding of ballast particles (E_S) and the breakage of ballast (E_B). The initial component, E_S, represents frictional sliding and is intricately linked to the imposed deviator stress (q_d) and the cumulative shear strain (ε_s). This factor considers the energy necessary to surpass the shear resistance originating from inter-particle friction. The subsequent term, E_B, correlates with the progression of particle breakage as determined by the BBI. Considering these factors, the subsequent empirical relationship is derived.:

$$E_S = a(q_d \varepsilon_s) \tag{1}$$

$$E_B = b(\kappa \times BBI)^c \tag{2}$$

where a, b, and c represent the parameters of the empirical model, a and c are dimensionless parameters, while parameter b is expressed in energy units. $\kappa = \sqrt{L_a / L_m}$ is the axle load factor normalized to the minimum axle load (L_m), as per the Australian standard for heavy haul operations, L_m= 20t, and L_a represents the simulated axle load in this study. For a standard 20t axle load (i.e. La = 20t, κ =1), b = 1 MJ/m^3 is associated with complete breakage (i.e. BBI=1) as indicated by Equation (2).

This energy dissipation model has been validated for up to 500,000 loading cycles. This value corresponds to approximately one year of typical heavy haul service in the State of New South Wales (NSW), involving an annual traffic tonnage of 50 million gross tons (MGT). Through non-linear regression, the model parameters have been calculated and are displayed in Figure 8, illustrating a strong correlation (R^2 >0.98) between the predicted and observed energy dissipation.

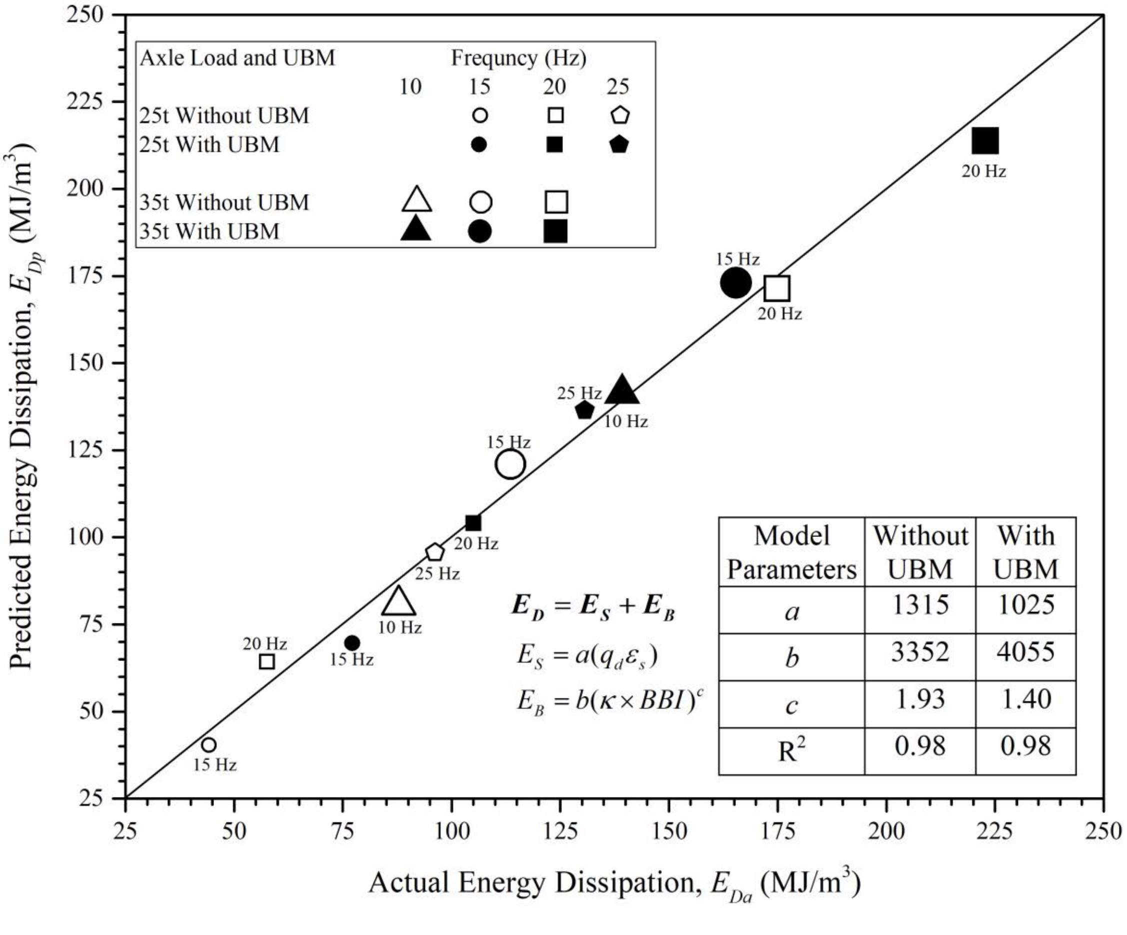

Figure 8: Energy dissipation model (modified after Navaratnarajah and Indraratna, 2017)

5 USE OF RUBBER TYRES AND TYRE SEGMENTS IN TRACK STABILISATION

This section introduces a hybrid track by combining two types of recycled tyre products consisting of (1) recycled tyre cells infilled with compacted recycled ballast to replace the usual capping layer of the railroad and (2) arch-shaped tyre segments from off-the-road (OTR) truck tyres installed in the shoulder ballast to reduce ballast lateral movement (Figure 9a) (Indraratna et al., 2022a). This technology results in tightly contained infill material and higher track-carrying capacity by utilising the 3D cylindrical form of the tyre cells and the damping property of rubber. In addition, the reinforced shoulder ballast using the OTR elements increases the track stiffness, reduces ballast layer deformations, and improves the load-bearing capacity of the track substructure (Sun et al., 2020).

5.1 TEST SETUP AND LOADING PROGRAM

The performance of the hybrid track was examined through the National Facility for Cyclic Testing of High-speed Rail. The trial track has a 4.1 x 4.1 m^2 cross-section and was made up of layers of various materials and depths (Figure 9b). Totally 16 waste rubber tyre cells (1 m in diameter and 275–300 mm deep) that had been filled with recyclable ballast were used to build the capping layer (Figure 9c) and a 300 mm-thick ballast layer was placed on top. To keep the material in the shoulder ballast contained, 4 arch-shaped OTR segments were also used. A complete track is shown in Figure 9d. More details about the test set-up and materials used can be found in Indraratna et al. (2022a).

Several sensors and instrumentation were installed on the test track to record data during the test. Settlement pegs and extensometers were used to measure the track's settlement and lateral displacement. Pressure plates were installed at different depths to record stress at various interfaces. Strain gauges were used to measure mobilised strain in tyre cells. Track accelerations at the sleeper and rail levels were measured using accelerometers. Ballast breakage was assessed using BBI by sieving it before and after the test. The test was conducted under a 25-tonne axle load, simulating a typical Australian freight train (vertical load of P_{max}=125 kN, P_{min}=15 kN, and P_{mean}=70 kN). A realistic range of heavy freight train speeds of 60–80 km/h on standard gauge lines was covered by the applied frequency of f=15 Hz. The test concluded after 500,000 cycles of load application.

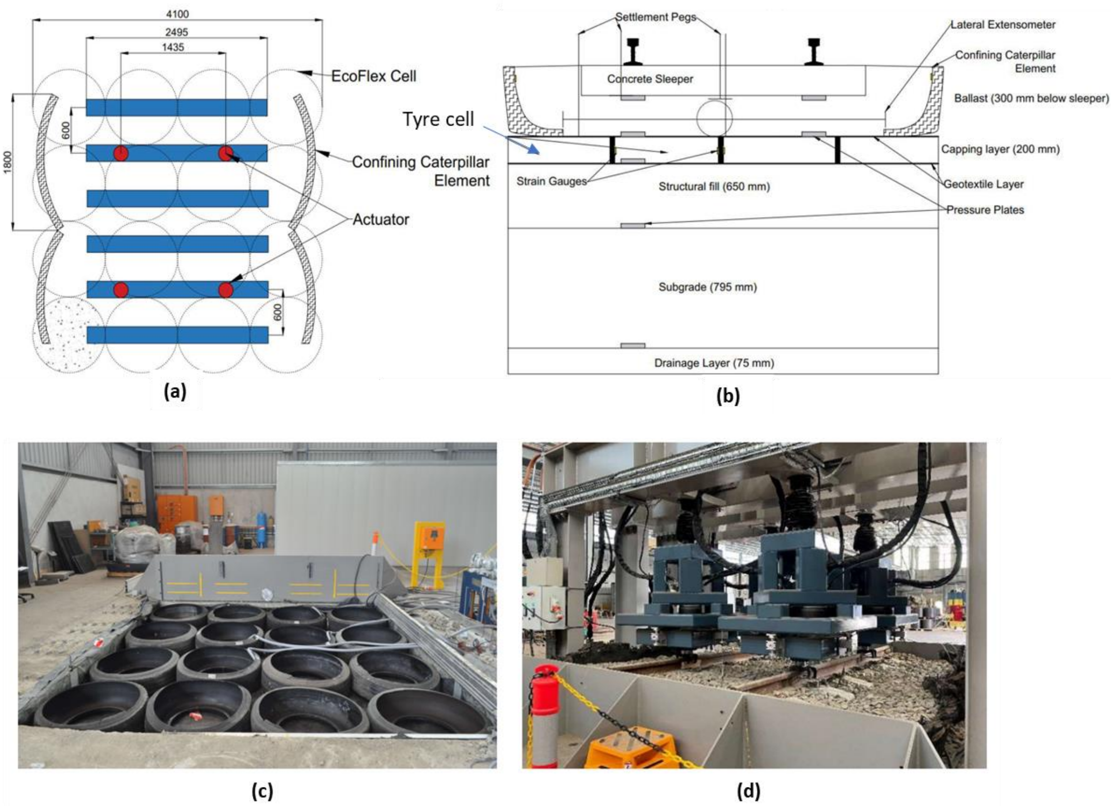

Figure 9: Schematic diagram of test track: (a) top view; (b) cross-section; (c) tyre assembly as capping; (d) complete track with shoulder-confined tyre segments (modified after Indraratna et al., 2022a)

5.2 MEASURED VERTICAL AND LATERAL DISPLACEMENT

Figure 10 shows the measured vertical and lateral displacement from this hybrid track and results obtained from a conventional track (Indraratna et al., 2021), laboratory test (Indraratna et al., 2013) and field measurements (Indraratna et al., 2010) are also included for comparison. It reveals that although the reinforced track initially settles more quickly than the unreinforced track, overall settlement on the reinforced way is lower. The reinforced track achieves stability significantly more rapidly than the unreinforced track, approximately N=100,000. The lateral displacement for both the unreinforced and reinforced track, as determined using lateral extensometers, is shown in Figure 10b. The lateral displacement in the strengthened track ranges from 3 to 6 mm, with an average displacement of 4 mm. In contrast, the unreinforced track displays a more significant lateral displacement reaching up to 9 mm. It can be concluded that the presence of shoulder reinforcement (OTR elements) in the reinforced track is responsible for the decrease in lateral displacement.

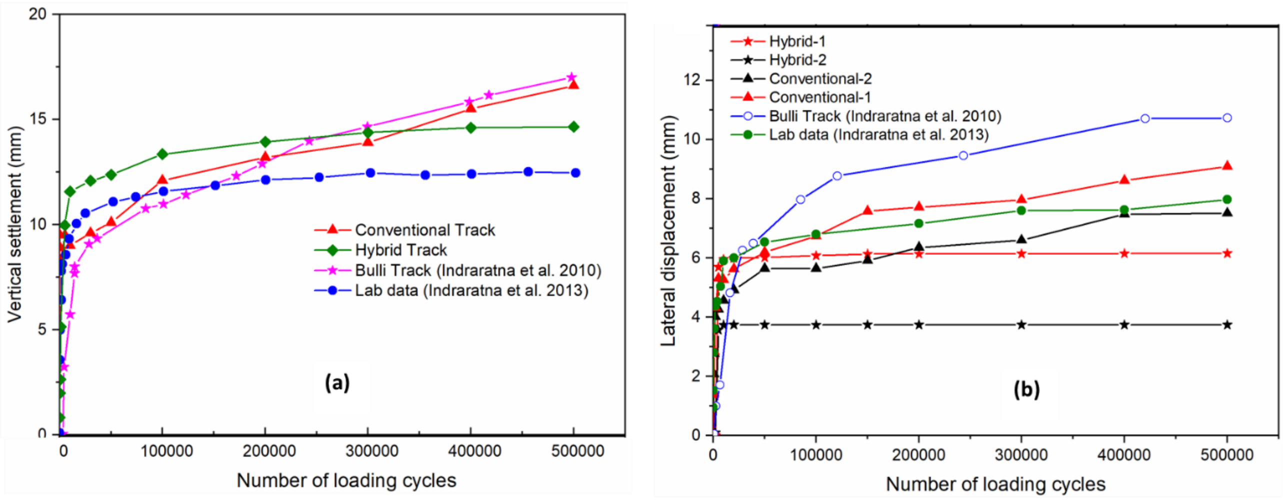

Figure 10: (a) Measured vertical settlement; (b) Lateral displacement of the hybrid track compared to a conventional track (modified after Indraratna et al., 2022a)

5.3 MEASURED STRESS DISTRIBUTION AND ACCELERATION

Figure 11 shows the measured stress distribution changing with depth for the hybrid track at different loading cycles and in comparison with results measured from the conventional track, laboratory and field tests. It is seen that as the number of loading cycles increases, the stress values at various depths decrease until N=100,000 loading cycles, at which point the stress values practically stabilise. The top ballast and capping layers take on more significant stress as a result of the reinforcement when compared to the unreinforced track stresses, and they transfer less stress to the underlying layers, which is highly advantageous in the case of soft subgrades (Indraratna et al., 2022a).

Figure 11b compares the rail accelerations of a hybrid track with those of a conventional track at N=200,000 cycles, as reported by Indraratna et al. (2021). The accelerations on the hybrid track are much lower, less than half of those on the regular track. Maximum accelerations observed on the hybrid track's rail were about 2.47 m/s^2, compared to 5.60 m/s^2 on the conventional track, demonstrating that incorporating recycled rubber elements in the track substructure can reduce vibration by at least 50%. At the end of the test, samples of ballast were collected and sieved to determine the amount of breakage. Measured results demonstrated that the hybrid track shows 33% and 42% reductions in ballast breakage below the actuator and shoulder locations compared to a standard track.

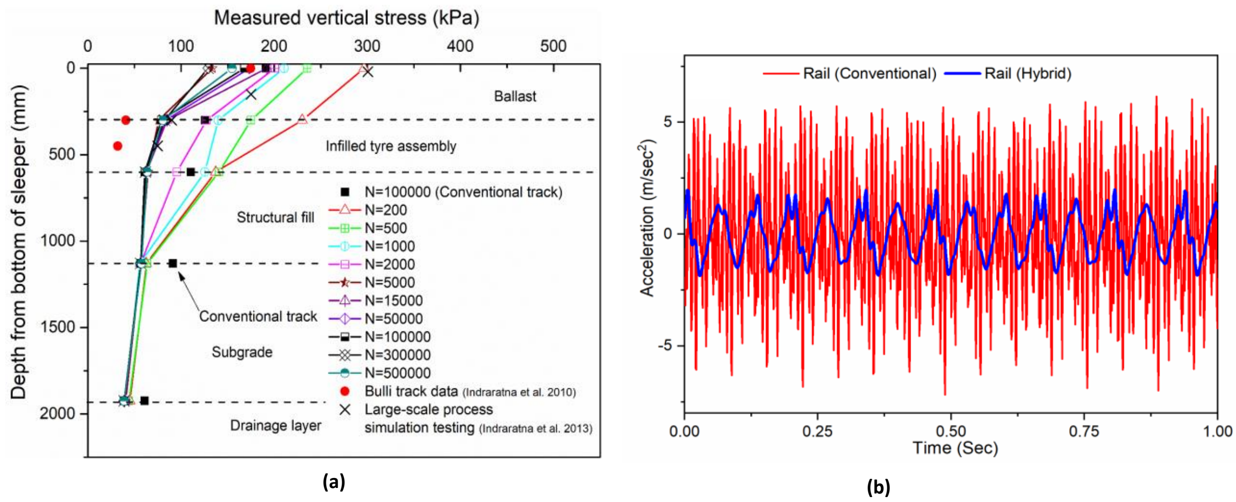

Figure 11: (a) Measured stress distribution with depth; (b) Measure accelerations on the rail (modified after Indraratna et al., 2022a)

6 CONCLUSIONS AND RECOMMENDATIONS

This paper reviews four state-of-the-art approaches to using recycled materials in transportation infrastructure including (1) using mining by-products coal wash and steel slag for port reclamation, (2) using rubber intermixed ballast system (RIBS) for minimising ballast breakage, (3) installing under ballast mats (UBMs) for reduced ballast deformation and degradation, and (4) employing a hybrid track of using recycled tyre cell infilled with recycled ballast in tandem with OTR segments for enhanced track performance. Large-scale laboratory tests and field trials were conducted to investigate their performance, and the following findings can be drawn from the above research:

- The coal wash and steel furnace slag blends were optimised using geotechnical testing, where the optimal blends should contain 35-60% coal wash by weight. A field test confirmed that average equivalent in-situ CBR values were between 25 and 60, similar to medium to dense sandy fills.
- The large-scale triaxial tests revealed that increasing rubber content in RIBS reduced the ballast dilation and shear strength and significantly mitigated the ballast breakage but caused the axial strain to increase. An optimal rubber content in RIBS was determined as 10% by weight as with this amount of rubber, RIBS had a substantial reduction (around 70%) in particle breakage while a minor reduction (<6%) in shear strength and acceptable axial strain compared to pure ballast.
- The hard interface beneath the ballast layer exerts a noteworthy impact on the overall deformation and breakage of the ballast. The incorporation of UBM on the surface of a concrete base plays a substantial role in diminishing both deformation and degradation. The test results obtained from large-scale cubic triaxial tests indicated that placing the UBM atop a concrete base (hard subgrade condition) could help to reduce the vertical deformation by approximately 10-20%, lateral dilation by around 5-10%, and ballast breakage by 35-45%. The impact of the incorporation of UBMs was also calibrated and verified by the proposed energy dissipation model.
- The overall accumulated lateral displacement and settlement of the hybrid track over a large number of loading cycles (N = 500,000) was smaller than the standard track. This finding implies that the use of recycled tyres in the track can provide immense benefits in relation to increased track stability and extended longevity of the hybrid track.
- Compared to an unreinforced track, the hybrid track significantly reduced the vertical stress distribution in track substructure layers, and reduced vibrations and this resulted in lesser ballast breakage. The results from this hybrid track test can lead to a conclusion that the use of recycled rubber in track not only reduces quarrying, and carbon emissions but also offers eco-friendly track stabilisation with an extended life cycle, and lower maintenance costs.

7 ACKNOWLEDGEMENTS

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CASE STUDY COMPARING EMBODIED CARBON EMISSIONS IN TWO ROAD-OVER-RAIL BRIDGE FOUNDATION DESIGNS

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ABSTRACT

This paper examines and compares the embodied carbon emissions in the earthworks and foundation design for two road-over-rail integral bridges as part of a project in Western Australia. The first bridge is supported on gravity footings and constructed using bottom-up methods, the second on load-bearing contiguous piles and constructed using top-down methods. The abutments support a cut profile of sand and limestone up to 10 m high.

A Life Cycle Assessment (LCA) for embodied carbon emissions was carried out for each bridge using the framework of PAS 2080. Construction-stage design information was used in the assessment, representing a bottom-up LCA approach to retrospectively identify carbon hotspots to inform future designs.

The assessment incorporated geotechnical site investigations; temporary works; bulk excavation for gravity footings and other minor excavations; and the raw materials for construction of the two foundation types. Transportation of materials to site, construction processes and final deconstruction and disposal of the structures were also considered. The bridge superstructure was outside the scope of the assessment.

The results are presented in total tonne CO_{2e} per bridge and tonne CO_{2e} per bridge deck area to allow direct comparison of the embodied carbon emissions of the two bridge foundation systems. The carbon hotspots in each design are identified, and the authors discuss how the results can be communicated to clients and contractors to be weighed alongside the various other drivers that influence construction method and design.

The paper closes with the authors' assessment of opportunities across the design process where geotechnical designers have most influence on embodied carbon over the design life of these bridge types.

1 INTRODUCTION

A rail extension project in Western Australia includes construction of bridges, stations and retaining walls along the alignment using a combination of top-down (such as contiguous piled walls) and bottom-up (gravity footings) methods. Due to the constraints of existing road levels and proposed development levels the rail alignment is founded in sections of cut.

The site is located within a coastal dune system, with subsurface units mainly comprising Tamala Sand and Tamala Limestone. The Tamala Limestone weathering profile has varying degrees of cementation and strength occurring within short lateral and vertical distances. Groundwater is approximately 20 m below foundation level, and the risk of karstic features varies from low to medium across the alignment. Karstic features such as solution features and cavities have been encountered during construction.

The overall cut to fill balance across the site is approximately 10:1. Excess fill is distributed to local developers and a nearby motorway upgrade project which intersects the site.

A Life Cycle Assessment (LCA) for embodied carbon emissions has been undertaken for two bridge sites within the project, one constructed top-down (Bridge No.1) and the other bottom-up (Bridge No.2), for comparison of embodied carbon emissions generated by the two foundation design solutions. The software One Click LCA was used for the assessment.

A design life of 120 years is applicable to both bridges. The bridges were also designed to require minimal maintenance, for example by adopting abutments integral with the superstructure to omit the requirement for maintenance of bearings.

It is the authors' opinion that LCAs for bridge design options typically focus heavily on superstructure components, whereas embodied emissions from earthworks and foundations systems are less well understood. Tools and software created for these assessments do not well represent geotechnical input requirements. Input data quality can also have a significant effect on the outcome of the LCA.

For the current case study, an initial assessment was carried out using construction stage information such as pile schedules, concrete and reinforcement drawings, and earthworks quantities estimated from drone imagery. The results of this initial assessment were compared against an LCA carried out using inputs obtained from a Bill of Quantities (BoQ). The outcome demonstrates some sensitivity to data input assumptions, which presents a challenge to geotechnical engineers working through these assessments who may not typically influence the structural components and may not always have ready access to the BoQ and 3D models. Further discussion is provided below.

2 BRIDGE NO.1 (TOP-DOWN CONSTRUCTION)

Bridge No.1 is a single span, two lane integral bridge constructed using top-down methods. The span is approximately 24 m wide and length 20 m (width is 35 m including approach slabs). The bridge abutments comprise load bearing contiguous piles socketed into limestone. The superstructure is integral with the abutment walls, and the wing walls are structurally independent contiguous piled walls. The bridge deck is an in situ concrete post-tensioned beam. Retained heights are in the order of 9.5 m, and original ground level is near the final road level.

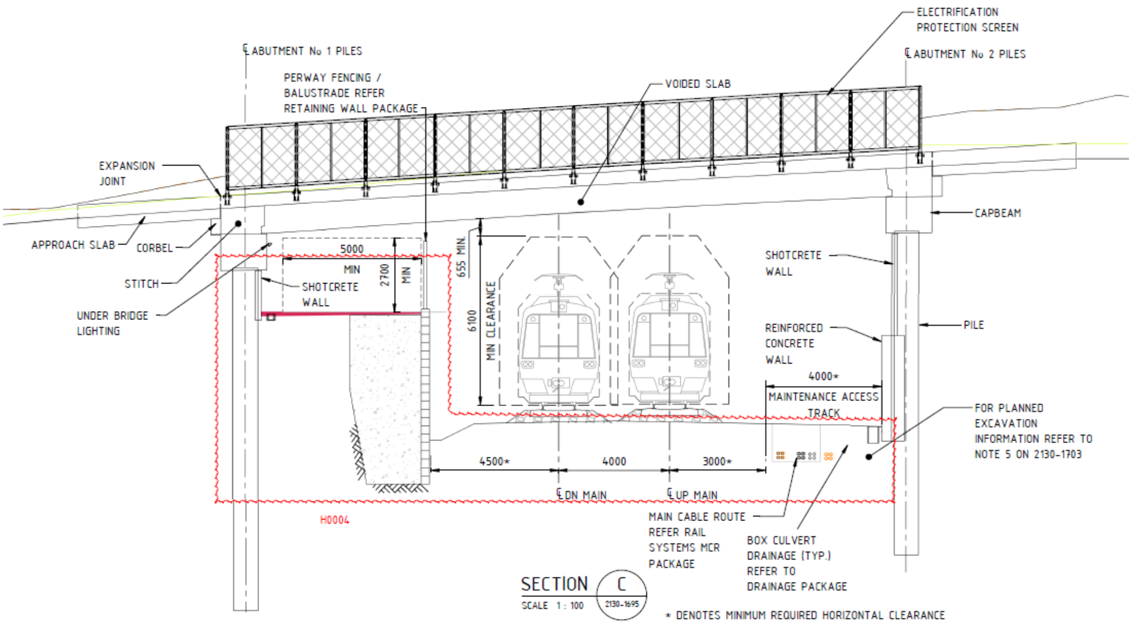

Figure 1: Bridge No.1 profile

Figure 2: Bridge No.1 during bulk excavation works

Bridge No.1 is founded on 900 mm diameter bored concrete piles, up to 20 m length and forming a contiguous piled wall. Pile design was carried out in accordance with Australian Standard AS 2159 and PD 6694-1:2011. An observational approach to piling was adopted using decision-trees to inform pile length based on materials encountered during piling, and some piles extended below design depth by up to 2 m where poor ground conditions (including inferred karstic features) were encountered. The total meterage of piling was in the order of 1,650 m.

Geotechnical investigations for the bridge comprised boreholes to about 20 m depth with Standard Penetrometer Testing (SPT) in sand and weak rock layers, and rock samples obtained for testing. The drilling method included a combination of rotary mud, diameter core drilling and air core drilling. Cone Penetrometer tests (CPTs) encountered shallow refusal on inferred rock. A geological cross-section through the bridge profile is provided below.

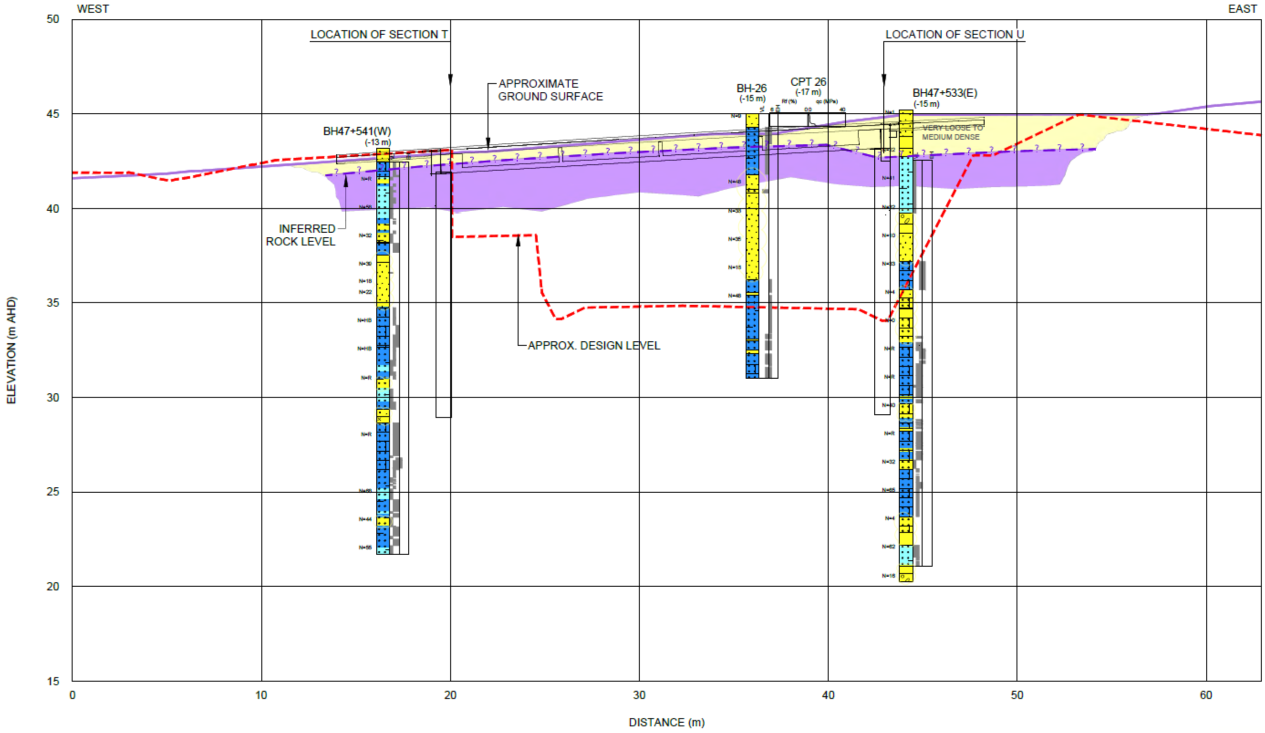

Figure 3: Geological profile at Bridge No. 1

During construction, a geotechnical engineer travelled to site for piling pad assessment and piling supervision. Pile Dynamic Analysis (PDA) with CAPWAP analysis was carried out on pre-production sacrificial piles to confirm load bearing capacity of the abutment piles. Several piles were subject to Thermal Integrity Testing (TIP). Concrete loss in three piles, inferred due to karstic cavities, triggered the requirement for targeted grouting works and Ground Penetrating Radar in the passive resistance zone to mitigate against voids affecting wall performance. Movement monitoring during bulk excavation was carried out using In-Place-Inclinometers installed within representative wing wall pile cages and steel survey pins installed along the capping beam.

Double pile geometry (rear piles spaced intermittently behind front piles) was required at wing walls to reduce wall deflections during bulk excavation works, necessitating a widened capping beam as shown in Figure 4 and 5 below. Reinforcement requirements differed between abutments and wing walls.

Temporary works included the requirement for additional piles at the wing wall extents to form return walls for construction of adjacent bottom-up structures. The piling platform and laydown pad comprised sub-base quality crushed limestone fill over site-won engineered sand fill compacted and tested in accordance with the project earthworks specification.

The permanent wall facing along the rail corridor will comprise a mesh reinforced shotcrete finish. In accordance with Australian Standard AS 5100, a reinforced concrete deflection wall is required along the eastern abutment within 10 m of the rail.

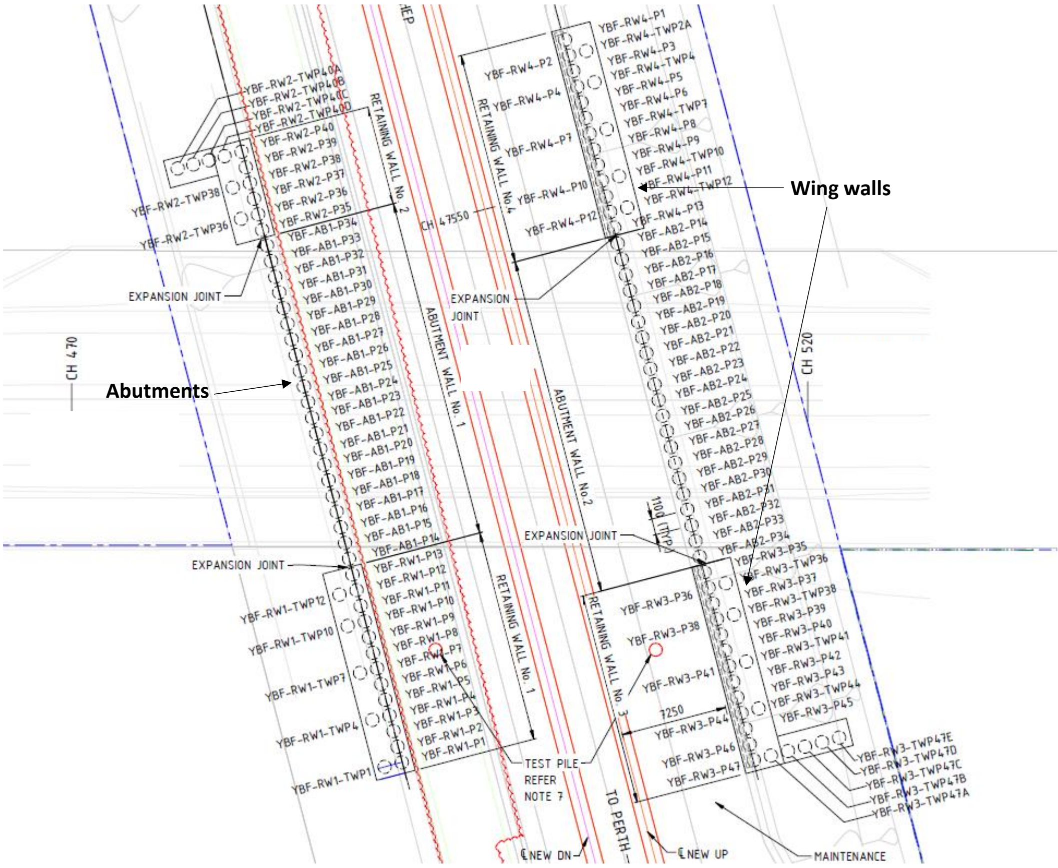

Figure 4: Bridge No.1 pile layout

Figure 5: Bridge No.1 pile capping beam excavation

3 BRIDGE NO.2 (BOTTOM-UP CONSTRUCTION)

Bridge No.2 is a single span, two lane bridge constructed using bottom-up methods. The bridge superstructure comprises precast beams with an in-situ topping slab made integral with full height reinforced concrete abutments, founded on shallow footings. The retained heights are in the order of 10 m, and footing widths up to 8 m. The span is approximately 20 m wide and the length 40 m (width is 35 m including approach slabs). The original ground level is above footing level.

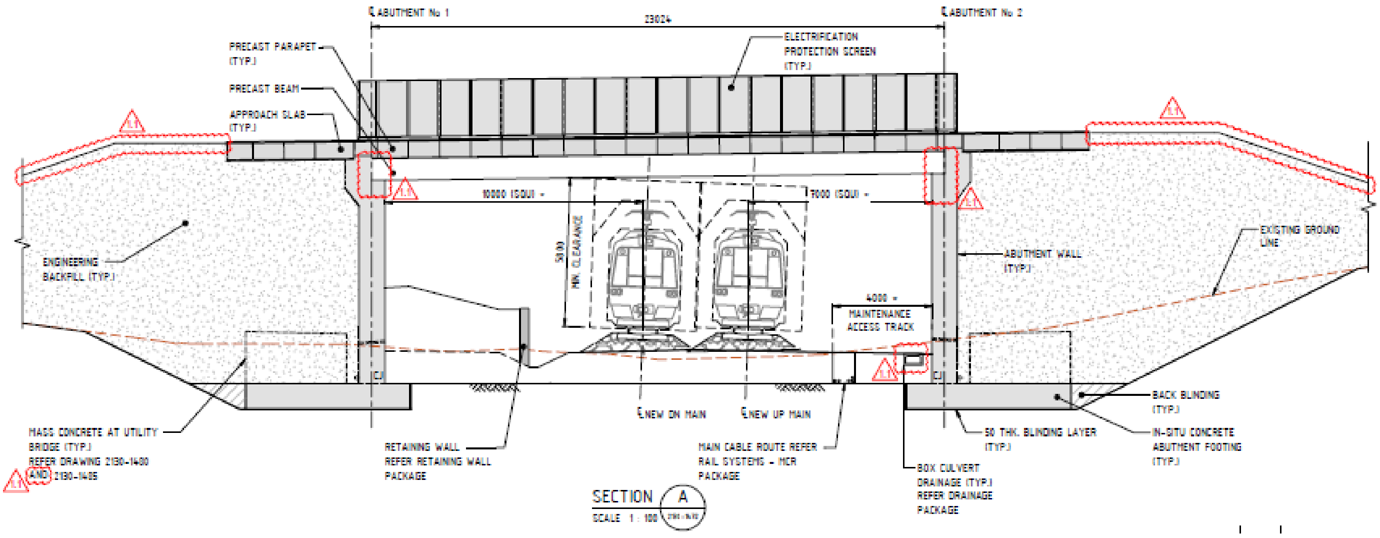

Figure 6: Bridge No.2 profile

Figure 7: Bridge No.2 during construction

The bridge foundations comprise reinforced concrete cast-in-situ L-shape wall footings varying in width. Wing wall footings are generally wider than abutment footings.

Geotechnical investigations comprised boreholes to an average depth of 25 m with SPT using rotary mud and diamond core drilling, with soil and rock samples obtained at regular intervals for testing. Vertical seismic profiling was carried out in two of the boreholes. A suite of soil and rock laboratory tests was carried out. CPTs were carried out to an average depth of 4 m. A geological long-section through the bridge profile is provided below.

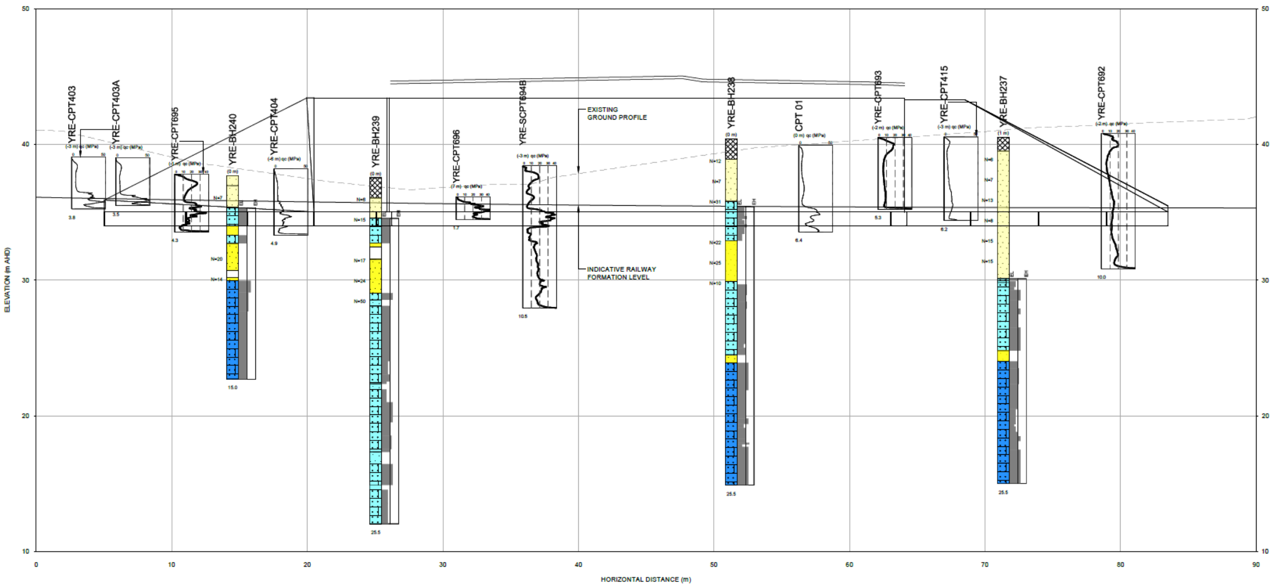

Figure 8: Geological profile at Bridge No. 2

Engineered backfill material behind the wall comprises site-sourced Tamala Sand, compacted and tested in accordance with the project earthworks specification. A geotechnical engineer travelled to site during construction for foundation inspections.

To reduce differential settlements across the footing over the sand /limestone interface, a compacted subbase quality crushed limestone foundation treatment layer was placed across the full length of the footing below blinding level. Steel survey pins were used for movement monitoring during bulk excavation works.

Figure 9: Bridge No.2 footing excavation (across sand / limestone interface)

4 CONSTRUCTION CONSIDERATIONS

Some of the key drivers for the choice of top-down or bottom-up construction methods are summarised in the table below. To the authors' knowledge, the embodied emissions of the different bridge options were not an explicit driver in the choice of construction methodology.

Table 1: Construction methodology considerations

Construction method	Advantages	Constraints / Disadvantages
Top-down	— Smaller excavation footprint, can accommodate boundary constraints — Road able to be opened prior to bulk excavation under the bridge	— Only practical where ground level is near final level — Piling rig and cage availability — Access to rail formation level available end of construction
Bottom-up	— Appropriate where ground level to be raised up from existing — Early access to rail formation level for installation of utilities, rail capping and ballast etc	— Larger excavation footprint, may not be possible (without temporary works) depending on boundary constraints — Earthworks requirement for excavation of material and placement and compaction of approved fill behind walls

Mechanically stabilised earth bridge abutments were considered as an alternative to bottom-up reinforced concrete L-shape walls, however, were not selected on the basis of durability considerations over the 120-year design life of the bridge. Reinforced earth structures can have lower embodied carbon emissions; however, the relative benefit should be assessed over the life of the structure as the design life may be less.

5 LIFE CYCLE ASSESSMENT METHODOLOGY

5.1 METHODOLOGY AND ASSUMPTIONS

An LCA for the two bridges was carried out using the guidance of PAS 2080. Modules A0-5 (Before Use Stage) and C1-4 (End of life stage) were assessed. It was assumed because of the integral nature of the sub and superstructures, the maintenance requirements for both structures would be minimal and therefore were not assessed. Items assessed for each stage are summarised in Table 2 below.

Construction-stage design information was used in the assessment, representing a bottom-up LCA approach to retrospectively identify carbon hotspots to inform future designs. Ideally these assessments are done early in the project to be able to influence design decisions (a top-down LCA approach).

Emission factors were obtained from verified Environmental Product Declarations (EPDs) only, provided through One Click LCA software. An exception is fuel consumption rates and fuel emissions factors for geotechnical site investigations and inspections, which were obtained from the SiteWiseTM Tool for Green and Sustainable Remediation.

Table 2: Items assessed for different PAS 2080 stages

PAS 2080 Module	Description	Items assessed
A-0	Preliminary studies, consultations	Geotechnical site investigations (both bridges), and geotechnical site inspections. Laboratory testing excluded
A-1	Raw material supply	**Bridge No. 1** (top-down) substructure:
A-2	Transport	

A-3 A-4	Manufacture Transport to work site	- Concrete for piles (abutments and wing walls), including test piles - Steel reinforcement for piles (abutments and wing walls) - Concrete and steel for shotcrete facing including deflection wall - Engineered fill for piling pad construction and laydown area - Engineered fill backfill for capping beam excavation - Bulk excavation for rail formation - Grout for infill of karstic cavities in front of piles **Bridge No. 2** (bottom-up) substructure: - Concrete for gravity footing - Steel reinforcement for gravity footing - Bulk excavation for gravity footing and rail formation - Engineered backfill behind the abutment and wing walls - Foundation treatment
A-5	Construction /installation processes	Calculated within One Click LCA software based on m^3 of removed / imported masses
B1-9	Use stage	Not assesessed
C1-4	End of life stage	Calculated within One Click LCA software as follows: "This includes impacts for processing recyclable construction waste or …processing waste streams that cannot be recycled. Additionally deconstruction includes emissions caused by waste energy recovery"

The assessment excluded the following items: embodied carbon of the bridge superstructure; greenhouse gases other than carbon dioxide; formwork and falsework required for construction; minor elements such as drainage, movement monitoring instrumentation and geotechnical investigation consumables; geotechnical laboratory testing, and pile integrity testing items. Travel to and from site by personnel other than geotechnical engineers was also excluded.

Transportation distances were calculated on the basis that pile reinforcement cages were shipped from eastern to western Australia, whereas other steel and concrete were sourced locally. Excavated soils were assumed to be transported for use by local projects, whereas imported soils were assumed to travel a greater distance to site. Based on the overall cut to fill balance for the site, it was assumed that 10% of excavated materials were re-used on site, with the remaining transported offsite. A bulking factor of 15% was applied to excavated sands and gravels.

6 RESULTS OF ASSESSMENT

Outcomes of the assessment are summarised in Tables 3 and 4.

Table 3: Embodied carbon emissions, Bridge No.1 (top down) substructure

PAS 2080 Module	Description	Design items assessed	Total CO_2e (t)	% of total
A-0	Preliminary studies, consultations	Geotechnical site investigations	20	1.5%
		Geotechnical site inspections/ pile supervision	0.9	<1%
A-1	Raw material supply	Design inputs		
A-2	Transport	- Abutment piles	258	**20.6%**
A-3	Manufacture	- Abutment capping beam	43	3.4%
A-4	Transport to work site	- Abutment shocrete facing	54	4%
		- Wing wall piles (60% steel/ 40% concrete)	463	**37%**
		- Wing wall capping beam	95	7.6%
		- Wing wall shotcrete facing	34	2.7%
		Construction inputs		
		- Abutment pile extensions, overbreak etc	35	2.8%
		- Wing Wall pile extensions, overbreak etc	100	8%
		- Wing wall capping beam extension	10	<1%
		- Grout for karst backfill	4.2	<1%
		Soils removed and sourced for project	75	6%
A-5	Construction processes	Earthworks operations for 21,210 m^3 of soils removed or placed on site	30	2.3%
B1-9	Use stage	Not assesessed		
C1-4	End of life stage	Waste processing and emissions during demolition	30	2.4%
			TOTAL = 1,252 (~1.25 t/m^2 of deck area)	

Table 4: Embodied carbon emissions, Bridge No.2 (bottom up) substructure

PAS 2080 Module	Description	Design items assessed	Total CO_2e (t)	% of total
A-0	Preliminary studies, consultations	Geotechnical site investigations	33	1.8%
		Geotechnical site inspections	0.1	<1%
A-1	Raw material supply	Steel and concrete for gravity footings (60% steel/ 40% concrete)	1,410	**80%**
A-2	Transport			
A-3	Manufacture	Soils removed and sourced for project	210	12%
A-4	Transport to work site			
A-5	Construction processes	Earthworks operations for 46,285 m^3 of soils removed or placed on site	65	3.7%
B1-9	Use stage	Not assesessed		
C1-4	End of life stage	Waste processing and emissions during demolition	41	2.3%
		TOTAL = 1,759 (1.37 t/m^2 of deck area**)**		

The quantities in Tables 3 and 4 were assessed first using construction stage information such as pile schedules, concrete and reinforcement drawings, and earthworks quantities estimated from drone imagery (within the battery limit of the project and extending only within the bridge footprint). The results were then refined based on more accurate assessment from BoQ for steel quantities at both bridges, which resulted in reduction in estimated steel quantities in the order of 10% to 20% and overall reduction in assessed emissions for both bridges in the order of 4% to 7%. This correction did not change the assessed relative emission intensity of the two bridge types or the overall insights from this study.

7 INSIGHTS GAINED

The results of the assessment indicate the bridge substructure constructed using top-down piling methods (Bridge No.1) generated in the order of 30% less embodied carbon emissions than the bridge constructed using bottom-up construction methods (Bridge No.2). Emissions per m^2 of deck area were also lower for this option.

The highest embodied carbon emissions for both bridges were generated by the raw materials supply and manufacture for the main foundation elements (abutment and wing wall piles, and reinforced concrete footings). Steel reinforcement was the most emission intensive material used in construction of the bridges, followed by concrete.

Geotechnical investigations accounted for up to 2% of bridge substructure emissions. However, had investigation not been undertaken at Bridge No.1, a rock socket could not have been assumed in design (given ground variability across the project) and piles may have been lengthened by an estimated 4 to 5 m each to accommodate a sandy rock socket design. This would have added approximately 500 m of pile length, equating to about 260 tonnes or an additional 20% of carbon emissions to the life cycle of the bridge. For Bridge No.2, sandy foundation conditions were confirmed by site investigation and assumed in design, so in this case site investigations did not materially affect the carbon emissions over the life cycle of the bridge (though were required for design input). Overall, the study demonstrates that site investigations can significantly reduce carbon emissions by providing greater design confidence to allow optimisation.

An observational approach to pile design at Bridge No.1 in the variable sand/rock conditions also prevented the design from being overly cautious and allowed piles to be lengthened only as required where sand pockets were encountered. In this instance, geotechnical design optimisation based on site investigation data and an observational approach to piling represented the best opportunities for the geotechnical engineers to influence embodied carbon over the design life of Bridge No.1. There was limited opportunity to optimise the design of Bridge No.2 and reduce embodied carbon emissions based on geotechnical conditions.

Construction processes (earthworks) accounted for only 2% to 4% of the bridge substructure emissions, where travel to source or remove soil offsite accounted for another 6% to 12%. Construction stage changes to the design (such as additional piles for temporary stability, pile extensions for poor conditions, pile overbreak etc.) accounted for an additional 15% of carbon emissions at Bridge No.1. Construction stage changes at Bridge No.2 did not materially affect carbon emissions to the authors' knowledge.

Transportation of raw materials (other than soils) accounted for up to 10% of emissions. Had the pile steel reinforcement cages for Bridge No.1 been able to be sourced locally within Western Australia, carbon emission could have been reduced by about 2% to 3% for the bridge substructure. There was an awareness among the design team of steel cage length limitation to fit on one truck bed, however the design requirements did not allow the pile lengths to be reduced below this length. This understanding was used elsewhere on the project however for design optimisation of alignment retaining walls. Travel distances for imported soils had limited effect on the overall assessment, as the bulk of the backfill was sourced on site. However, Bridge No. 2 assessment was sensitive to assumed truck movements within the site for backfilling works, where every 1 km of additional distance to/from soil stockpile areas represents 1.5% of the embodied emissions for the bridge (over the total number of truck movements).

Items not considered to be of material significant to the assessment included geotechnical inspections and small items such as the grouting program for karstic features.

From these insights it is apparent that familiarity with LCA for different bridge foundation solutions will allow geotechnical and structural engineers to optimise their designs, and where possible, move towards lower embodied emissions solutions over time. Development of a consistent LCA assessment methodology across the geotechnical industry is however needed to facilitate easy comparison of different project examples. The current case study also indicates that foundation solutions that allow for an observational approach during construction and where construction constraints are well understood provide more opportunities for geotechnical engineers to optimise the design and ultimately reduce embodied emissions of the bridge structure, when compared to more conservative design approaches.

8 ACKNOWLEDGEMENTS

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EMBODIED CARBON ASSESSMENT OF GEOTECHNICAL WORKS

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ABSTRACT

In the light of rising construction sustainability concerns, embodied carbon assessments are often one of the main engineering tools to identify the best "green" option. Embodied carbon assessments provide a simple way to quantify and measure the summation of all the greenhouse gases generated from the built environment. It includes a whole life carbon cycle assessment of a given project from the impacts of materials production, transportation, installation, maintenance, and any waste or disposals during and at the end of design life. This paper aims to allow geotechnical engineers to quickly determine the embodied carbon of their design, and more profoundly form the basis of an innovative and efficient design approach with the consideration of intelligent and alternate material choice to achieve the same performance. In this paper, the methodology of embodied carbon calculation will first be introduced, followed by a summary of carbon emission factors (CEF) that are applicable for geotechnical designs. The discussion herein will focus on the initial portion of the embodied carbon life cycle assessment which comprises of the "before use stage" only for a particular project. Case studies on the use of embodied carbon calculations were provided for a variety of geotechnical projects including foundation for road embankment, trench excavations, and tunnel design. These case studies will show the significance of carbon calculations during the initial design stages and its value in recognition of projects' sustainability goals. Alternative real-life solutions in achieving de-carbonization will also be presented as a concluding remark, highlighting the possibility of sustainable design in geotechnical practice.

KEYWORDS: Embodied carbon assessment, carbon emission factors, de-carbonization

1 INTRODUCTION

The Paris Agreement (COP21) signed in 2015 by 196 members of the United Nations is a legally binding treaty with a goal of limiting temperature increases to 1.5°C. In 2018, the Intergovernmental Panel on Climate Change (IPCC) reported that in order to limit temperature rise to this level, global carbon dioxide emissions needed to be reduced by 45% by 2030, and net zero by 2050. These global policies helped to change the industry's practice by increasing the awareness of global warming, and particularly the impact of carbon. The embodied carbon (EC) assessment helps the designer quantify the amount of embodied carbon in their design and promotes sustainable strategies in the construction industry. As a result, it has become crucial to carry out quick embodied carbon calculations that align with the Sustainable Development Goals (SDGs) (United Nations, 2015). EC calculations particularly align with the following SDGs: No. 9 (Industry, Innovation, and Infrastructure), No. 12 (Responsible Consumption and Production), and No. 13 (Climate Action).

The authors are of the opinion that most Geotechnical Engineers either don't complete or leave the embodied carbon (EC) calculations up to their Civil/Structural Engineering colleagues. Whilst Geotechnical Engineers typically refine or optimize their designs to reduce quantities and costs, most lack the understanding (or time) to undertake the EC calculations themselves. This paper aims to close the gap and provide the motivation and means for Geotechnical Engineers to quickly undertake their own calculations and hopefully fuel further discussion of sustainability in Geotechnical Engineering.

A whole life carbon assessment typically encompasses all stages and is also commonly referred to as a 'cradle to grave' assessment. From PAS 2080:2016 (BSI, 2016) and RICS (2017), the whole life carbon cycle is broken down into three main stages, known as 'Before use' (Stage A), 'Use' (Stage B), and 'End of life' (Stage C) stages, as shown in Figure 1. Stage D is typically not considered in a simple embodied carbon calculation. The carbon assessment is further categorised by the source of the emissions into capital carbon, operational carbon, and user carbon (EFFC, 2022). In the context of a whole life carbon assessment and 'embodied carbon calculation', the amount of greenhouse gas emissions is typically measured in the unit of carbon dioxide equivalent (CO_2e) or as a mass factor of $kg.CO_2e$. This allows quantification of the global warming potential and provides a basis for an optioneering assessment. The methodology and associated formulae for this kind of assessment is given in Section 2, followed by a summary of the selective carbon emission factors (CEFs) for different stages in section 3.

The term embodied carbon (EC) in the context of this paper is used to represent the capital carbon greenhouse gas (GHG) emissions associated with the 'Before Use' (Stage A) only, also known as a 'cradle to practical completion' assessment. It is understood that the term embodied carbon is also commonly used to represent a whole life carbon assessment. It's therefore imperative that the engineer defines the system boundaries of the 'embodied carbon' assessment. As designers, Engineers traditionally have the greatest ability to impact Stage A1-A3 (EC involved with the type and quantity of material), and construction companies Stage A4-A5 (EC involved with the transportation and installation on site). However, to meet the IPCC climate actions goals it's imperative that engineers start broadening their system boundaries to thinking whole-life and start working collaboratively with construction companies to reduce the embodied carbon of the built environment. Geotechnical specific embodied carbon case studies illustrating this type of thinking were given in section 4.

From a viewpoint of the carbon reduction hierarchy, the earlier the carbon assessment takes place the greater the ability the designer has to reduce the embodied carbon in projects and programme of work, as illustrated in Figure 2 (BSI, 2023). The adoption of low-carbon techniques are examples of 'improve' mitigations to reduce the whole life carbon in projects (refer Figure 2). Examples demonstrating 'avoid' or 'switch' techniques to reduce carbon will be given in Section 5, followed by some decarbonisation methods to 'improve'.

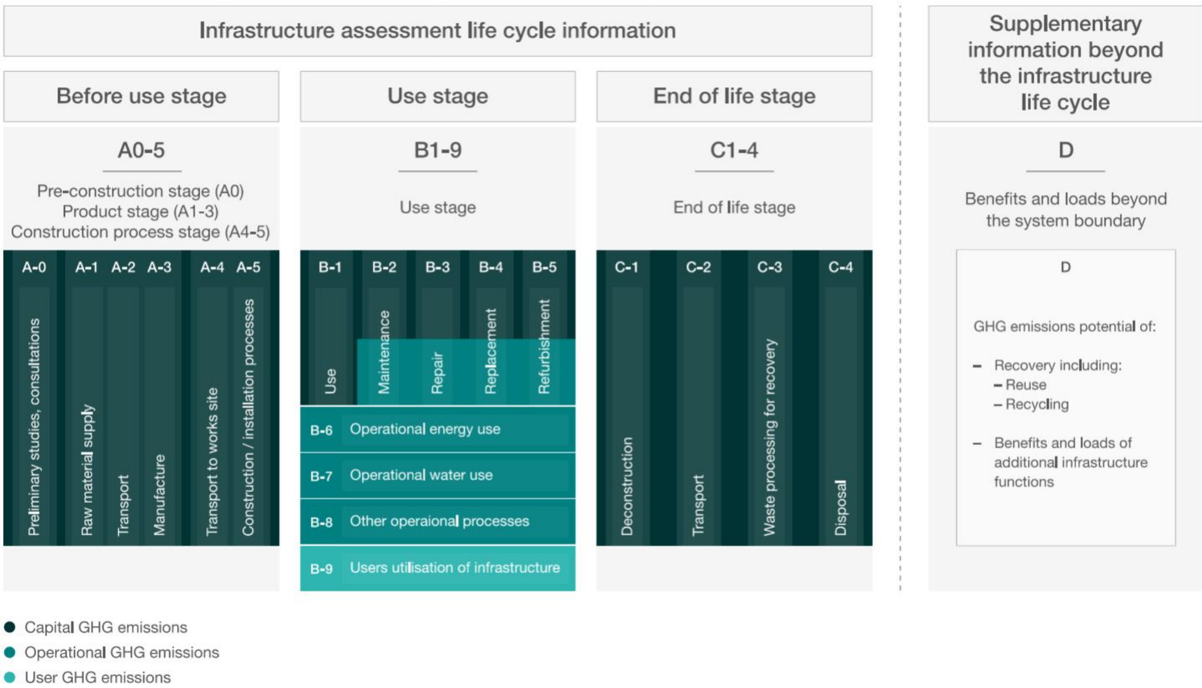

Figure 1: Whole life carbon assessment stages from PAS2080:2016 (BSI, 2016)

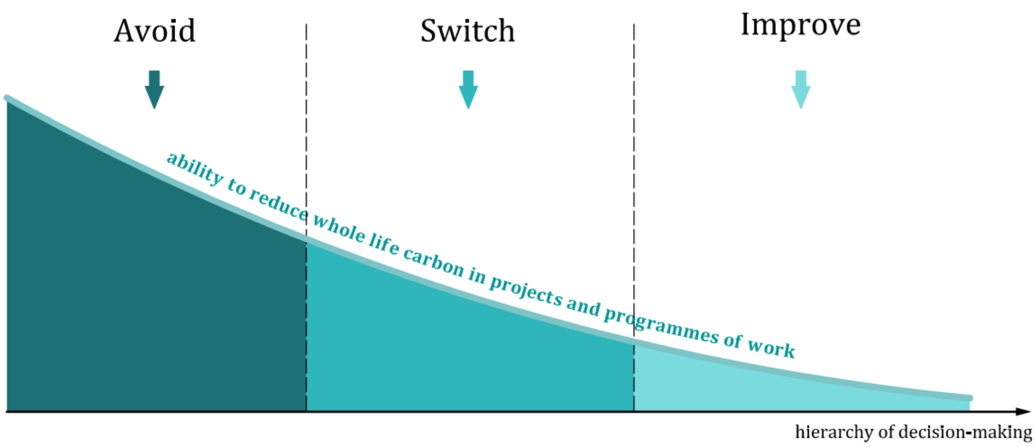

Figure 2: Carbon reduction hierarchy from PAS2080:2023 (BSI, 2023)

2 SIMPLIFIED EMBODIED CARBON CALCULATION

The word ‘simplified’ in the context of this paper is used to describe the extent of carbon study from Stage A1 to A5 only. The consideration of whole life carbon (Stages A-C) is not addressed herein.

The fundamental objective of embodied carbon calculations is to quantify the carbon impact from ‘cradle to practical completion’ of the building or infrastructure asset. This aids the engineer in comparing design options (i.e., optioneering), benchmarking, as well as target setting in carbon reduction (RICS, 2017). Benchmarking refers to the comparison between a project against itself over time (‘dynamic benchmarking’), and against other similar projects (‘static project’) under the same basis with consistent results. Ultimately carbon targets set the goal and precedence for carbon reduction. These targets could include the sustainable development policies or planning requirements for the project.

As summarized in Table 1 after BS EN 15978 (BSI, 2011), a typical embodied carbon (EC) assessment from ‘cradle to practical completion’ is classified into three main categories. A simple EC calculation involves multiplying a quantity, (such as material mass or volumetric quantity of fuel or electricity) by the corresponding carbon emission factor (CEF). The specific quantity is related to resource use which is work-specific depending on the scale and size of a project, whereas the latter (CEF) is a constant determined from public research or industry published data. The authors note that most engineers have a reasonable grasp of the quantities/volumes, but seldom know the appropriate CEF to apply in the EC calculations. Guidance on the selection of CEF are given in Section 3, with industry references sourced.

Table 1: Embodied carbon formulae for Stages A1-A5

Stage	Description	Embodied carbon formula (*kg.CO_2e*) after BS EN 15978 (BSI, 2011)
A1-A3	Material/product stage	EC = Material mass (*kg*) × $CEF_{material}$ (*kg.CO_2e/kg*)
A4	Transportation of material stage	EC = Material mass (*kg*) × Transport distance (*km*) × $CEF_{transport}$ (*kg.CO_2e/kg* per *km*)
A5	Construction process	EC = Project Cost ($) × $CEF_{construction}$ (*kg.CO_2e/kg* per $) or, EC = EC_{fuel} + $EC_{electricity}$ = volume of fuel consumed (*L*) × CEF_{fuel} + electricity consumption (*kWh*) × $CEF_{electricity}$

As illustrated in Figure 3 (BSI, 2023), whilst the early work stages provide the greatest opportunity to reduce whole life carbon (as discussed in Section 1), project uncertainty is also high. Therefore, the designer must make educated assumptions in order to undertake an EC calculation. It is for this reason that EC calculations (in the early design stages) lend themselves more to comparison purposes rather exact measurement. The true carbon values may fluctuate across the project work phases and should be evaluated once more data is available.

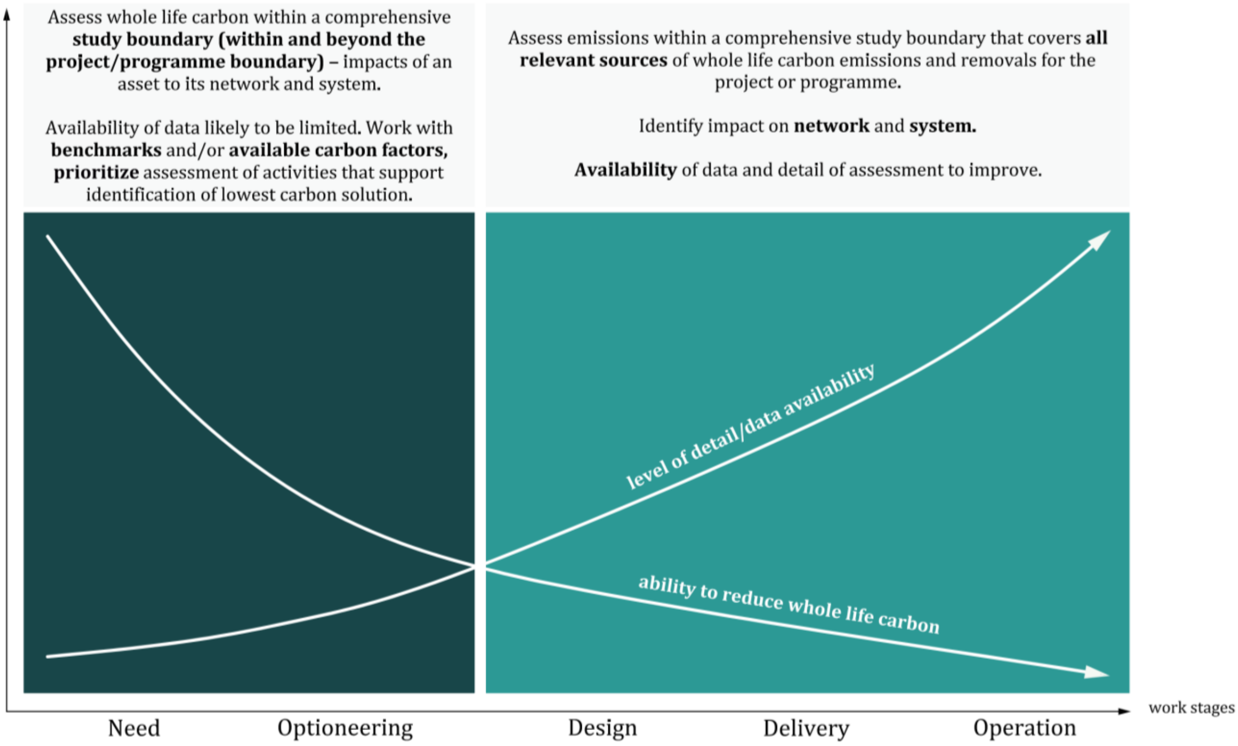

Figure 3: Plot illustrating data availability and the ability to reduce carbon in different work stages (BSI, 2023)

3 QUICK GUIDANCE ON THE SELECTION OF CARBON EMISSION FACTORS

Carbon emission factor (CEF), measured in $kg.CO_2e$ per unit, quantifies the amount of 'carbon' involved with the product or activity per unit. CO_2e, is a measure used to compare the emissions from various greenhouse gases on the basis of their global-warming potential (GWP), by converting amounts of other gases to the equivalent amount of carbon dioxide with the same global warming potential.

CEF's are material specific and typically vary across different countries for the same materials due to a variety of factors including different industrial practices and economic conditions. They are continually updated in line with current industry behaviours. Therefore, the most accurate CEF's for a material are typically provided by product specific, Environmental Product Declarations (EPDs). EPD's are an independently verified and registered document, adopted in different countries to quantify environmental impacts on the life cycle of a product (ISO, 2006). Various databases have been set up that collate local EPD's to provide country specific CEF's as well as worldwide average CEF's. Australian specific CEF's can be sourced from EPiC database (Crawford, et al., 2019) and UK and worldwide factors from the ICE database (Jones & Hammond, 2019). The full list of resources referenced in the production of this paper is provided below:

- Environmental Performance in Construction (EPiC) Database, University of Melbourne, (Crawford, et al., 2019).
- Inventory for Carbon and Energy (ICE) database (V3.0), University of Bath, (Jones & Hammond, 2019).
- Veracity (v0.1.4), Arup in-house carbon database (Arup, 2019).
- IStructE, 'How to Calculate Embodied Carbon' (Gibbons & Orr., 2020).
- Royal Institution of Chartered Surveyors, Whole life carbon assessment for the built environment (RICS, 2017).
- Product specific EPDs.

The following sections (Section 3.1 to 3.3) provide a summary of the recommended CEFs for common geotechnical project types. They were originally written from UK or European specific data and adapted for Australian factors where applicable to suit local practice. This section of CEF quick guidance was sourced from the documents of 'Geotechnical Embodied Carbon Cheat-Sheet (Aus)' (Dewar & Cheng, 2021) and 'Geotechnical embodied carbon cribsheet – Supplementary manual' (Dewar, 2021), which were prepared by Nick Dewar as part of Arup's investments and in-house resources. Factors should be selected with caution and assumptions documented, so that updates can be easily made as project uncertainty decreases.

3.1 MATERIAL STAGE (A1-A3)

The two main types of engineering materials (excluding soil) used in the construction industry (especially in geotechnical engineering) are concrete and steel. Typical CEF's for these materials are provided in the following tables, Table 2 to Table 4.

Other than the conventional form of CEF in the unit of *kg.*CO_2*e/kg*, it could also be represented by Carbon Factor in the unit of *kg.*CO_2e/m^3 with a known material density. The relation between them is given by:

EC (*kg.*CO_2*e*) = Material mass (kg) × $CEF_{material}$ (*kg.*CO_2*e/kg*) or,

EC (*kg.*CO_2*e*) = Volume (m^3) × Carbon Factor (*kg.*CO_2e/m^3)

Table 2: Suggested carbon factors for typical concrete mixes used in geotechnical structures

Typical uses	Concrete strength[1,2] (*MPa*)	Typical cement mix[1]	CEF[3] (*kg.*CO_2*e/kg*)	Carbon factor (*kg.*CO_2e/m^3)
Secant piles (primary)	C8/10 [4]	CEM III/A GGBS (50%) GEN 1 [5]	0.092	219
Blinding concrete	C20	CEM II/B-S 30% GGBFS	0.113	269
Piles (Bored, CFA), Pad footings, Retaining walls, Secant Piles (secondary), Contiguous/Solider walls, Diaphragm walls etc.	C32/40	CEM II/B-S 30% GGBFS	0.142	389
Precast concrete driven piles, king post walls etc.	C40/50	CEM II/B-S 30% GGBFS	0.163	389
Ground slabs, Pile Caps, Capping beams, Ground beams	C40/50	CEM II/B-S 30% GGBFS	0.163	389

Notes:
1. Typical mix and strength based on project experience
2. Concrete strength in accordance with AS3600:2018 (Australian Standards, 2018)
3. Data extracted from EPiC database (Crawford, et al., 2019)
4. Global value is adopted with reference to data from ICEv3 database (Jones & Hammond, 2019)
5. CEF sourced from ICEv3 database (Jones & Hammond, 2019). IStructE guide (Gibbons & Orr., 2020) also typically references ICE V.3 database.

Table 3: Suggested CEF's for typical steel elements used in geotechnical structures

Typical uses	Steel type	CEF (*kg.*CO_2*e/kg*)
Reinforcing steel in concrete structures	Steel rebar	2.10 [1]
		1.99 [2]
Helical piles & other CHS sections	Seamless tube	4.60 [1]
Driven steel piles, pile casing (& other welded steel tubes).	Welded pipe	3.50 [1]
	Steel plate	2.46 [4]
Sheet piles - hot rolled (Z-shaped, U-shaped, straight web and H-shaped)	Steel, Section	1.55 [4]
Sheet piles - Cold formed (omega-shaped, Z-shaped, trench sheets)	Steel, Finished Cold Rolled Coil	2.73 [4]
Steel H beams (king post wall) etc.	Steel, Section	3.30 [1]

Typical uses	Steel type	CEF ($kg.CO_2e/kg$)

Notes:
1. Data extracted from EPiC database (Crawford, et al., 2019). Recycling not considered.
2. European average data from worldsteel LCI, 85% recycling rate considered.
3. Structural steel manufactured in accordance with AS/NZS 1163; manufactured through an extrusion process.
4. Global average data from ICE v.3 (Jones & Hammond, 2019) which sources its data from worldsteel LCI (Worldsteel Association, 2019). Recycling not considered

Table 4: Typical steel reinforcement rates and corresponding carbon factors for geotechnical structures

Category	Options	Typical range		Chosen value for carbon calculation			
		Steel rate[1] (kg/m^3)	% of steel range[1]	Steel rate (kg/m^3)	% of steel	CEF[2] ($kg.CO2e/kg$)	Carbon factor[2] ($kg.CO2e/m^3$ of concrete)
Shallow foundation	Rafts: Ground bearing/shallow	115	1.5	115	1.5	1.99	229
	Rafts: Piled rafts in heaving ground	150-200	1.9-2.5	170	2.2	1.99	338
	Pile caps	110-150	1.4-1.9	120	1.5	1.99	239
Deep foundation	Bearing piles: Fully reinforced subject to heave unloading (0.75-2.1m dia.)	80-160	1-2	120	1.5	1.99	239
	Bearing piles: Partially reinforced not subjected to heave (0.45-1.2m dia)	20-80	0.3-1	50	0.6	1.99	100
Earth retention/ Basement	RC retaining wall (L-shaped, gravity etc.)	100-300	1.3-3.8	200	2.5	1.99	398
	Secant piled wall: Hard/firm (600-750mm piles) [3]	115-190	1.5-2.4	160	2.0	1.99	318
	Secant piled wall: Hard/firm (900-1200mm piles) [3]	100-150	1.3-1.9	120	1.5	1.99	239
	Contiguous piled wall (bored, CFA): For typical basements up to 8m depth	80-160	1-2	150	1.9	1.99	299
	Diaphragm wall (incl. guide wall)	130-180	1.7-2.3	150	1.9	1.99	299
	Guide walls	40-60	0.5-0.8	50	0.6	1.99	100
	Capping beams	180-220	2.3-2.8	200	2.5	1.99	398

Notes:
1. Typical steel reinforcement rates based on industry practice (eg. Arup experience), previous projects and review of Arup Structural Concept Design Guide, Concrete Society: Concrete Buildings Scheme Design Manual (section 3.7), F.Cobbs - Structural Engineers Pocket Book (concrete section).
2. European average data from worldsteel LCI. 85% recycling rate considered given that recycling rate for scrap steel is around 80-90% in Australia (Transport Canberra & City Services, ACT Government, 2018).
3. Steel reinforcement rate for secondary pile only (primary unreinforced).

Earth fill materials are also commonly used in geotechnical works. Below (Table 5) shows the recommended CEFs values for some typical earth fill types and asphalt. Some filling types may require pre-treatment to ensure their suitability in the earthwork. It should be noted that below CEFs do not capture the embodied carbon related to the material improvement processes and additional materials. More research should be done to accommodate the impacts of 'improved' materials on carbon footprint used in EC calculations.

Table 5: CEFs for typical filling materials

Fill types	Typical uses	CEF (*kg.CO_2e/kg*)	Source
Gravel (aggregates)	Landscaping, drainage layer	0.036	EPiC data (Crawford, et al., 2019)
General fills (sandy materials)	General earthworks, landscaping, free draining granular fills, drainage layer	0.024	ICE v3 database (Jones & Hammond, 2019) and EPiC data (Crawford, et al., 2019)
Recycled aggregates	Landscaping, drainage layer	0.008	EPiC data (Crawford, et al., 2019)
Asphalt, 5% of bitumen as binder content (by mass)	Road surface, pavement	0.054	ICE v3 database (Jones & Hammond, 2019)
Asphalt (general mix)		0.200	EPiC data (Crawford, et al., 2019)

3.2 TRANSPORATION OF MATERIALS STAGE (A4)

In the absence of local research, the following CEFs are taken from UK guidelines, as shown in Table 6 and Table 7. It should be noted that the travel distance here refers to the distance from the material factory to the designated project site. Hence, transport distances shown in Table 7 are indicative and should be taken only if actual distance is unknown. They are sourced from RICS **(2017)**, and adjusted for Australian conditions. The source regions are categorized into the following travel 'areas': 'local', 'national', 'regional (Australasia)' and 'global'.

Table 6: Typical CEFs for different transportation modes

Mode	CEF transport (*gCO_2e/kg/km*)[1]
Road transport emissions, average laden	0.1065
Road transport emissions, fully laden	0.07524
Sea transport emissions	0.01614
Freight flight emissions	0.59943
Rail transport emissions	0.02556
Notes: 1. Sourced from IStructE (Gibbons & Orr., 2020)	

Table 7: Typical CEFs for transportation (A4) for construction materials

Material	Sourced from region[1]	*km* by road[1]	CEF transport (*gCO_2e/kg/km*)[2]
Concrete	Locally manufactured	50	0.1065
Steel	Nationally and locally manufactured	600[3]	0.1065
Controlled fill (type 6N etc)	Locally manufactured	50	0.1065
Other: lime, bentonite/polymer, stone etc.	Locally manufactured	50[4]	0.1065
Geotextiles, geomembranes plastics etc	Locally manufactured	50[4]	0.1065

Notes: When undertaking A4 calculations best practice is to consider the return journey (i.e., travel to and from site). However, it is common to only consider a one-way journey. Either method is acceptable as long as the system boundaries in the calculation are documented.
1. Sourced from RICS (2017) and adjusted for Australian conditions
2. Sourced from IStructE (Gibbons & Orr., 2020), assumed road transport with average laden
3. Average mean by assuming approximately one-third of the steel production from national sources and the rest from local manufacturers
4. Subjected to higher travelling distance if materials are not available locally

3.3 CONSTRUCTION STAGE (A5)

The embodied carbon calculation for Stage A5 is difficult to quantify, especially at the design (or pre-design) stage when limited information is available about the construction sequences on site. The formulae given in Table 1 required the consumption usage of both electricity and fuel. Appropriate assumptions should be adopted to estimate these quantities, perhaps based on past project experience or similar project types. In addition, there are obvious limitations in taking extensive measurements of embodied carbon during construction works. The above calculation of energy consumption (also denoted by Stage A5a) did not capture the carbon emissions component due to labour resources such as the activities of concreting and formwork. The activities associated with waste disposal (Stage A5w) have also not been considered. Alternatively, the embodied carbon for waste can be captured in the A1-A3 (material production) stages by allowing for an additional quantity or volume of material. An example could be CFA piles where an overbreak of approximately 10-15% is typical for concrete consumption. Hence, more research and efforts from the contractors and design engineers to establish a proper database as an example of EC_{A5} inputs are recommended.

Another way of forecasting the EC_{A5} could be done by the estimation from project cost, suggested by RICS (2017) and IStructE (Gibbons & Orr., 2020). Caution should be made when adopting this approach, as the published correlations were for high-rise buildings and based on UK data only. Given that the studies from these institutes focus on the EC of building projects which have its project scale in proportion to project cost in most circumstances, the actual EC_{A5} for geotechnical projects computed by this method should be reviewed once more information of construction details are available. The CEF_{A5} for cost estimation approach are given in below table (Table 8) for reference only.

At the time of writing the authors are not aware of any similar published correlations for infrastructure assets or buildings in Australia.

Table 8: Embodied carbon rate for site activity emission for building construction.

Rate (*$kgCO_2e$* per £100*k*)	**Project constraints**
1400 [1524]	Construction cost for the whole building
700 [762]	Construction cost for the superstructure or substructure only
Note: [1524/762] 2020 rate, graded for inflation. RICS (2017) suggests a construction carbon emission factor of 1400$kgCO_2e$ per £100k construction cost for the whole building. IStructE (Gibbons & Orr., 2020) suggests a 50% reduction in the construction carbon emission factor to 700$kgCO_2e$ per £100k construction cost for superstructure or substructure only. Values are based on a 2015 assessment and should be adjusted in line with inflation.	

For quick computation, the authors suggest an alternative way of EC_{A5} estimation based on the assumption of fixed $EC_{A5}/EC_{A1\text{-}A5}$ ratio for each construction activity type. For instance, piling activity for one project should share similar $EC_{A5}/EC_{A1\text{-}A5}$ percentage to another project at different site despite of the resources' quantities spent, which was captured in the factor component of $EC_{A1\text{-}A3}$ and EC_{A4}. Hence, given the values from $EC_{A1\text{-}A4}$ determined in previous steps, the value of EC_{A5} could be back-calculated with reference to this fixed assumed ratio, whereas this ratio ($EC_{A5}/EC_{A1\text{-}A5}$) could be resolved by past project experience. A database of past project information is required for this approach of calculation. Example of this computation method was illustrated in Section 4.1. It should also be noted that this computed EC_{A5} is rough estimation only given the condition of data deficiency in early design stage. More profound research on EC_{A5} and updates in computed values based on more available details is recommended.

4 APPLICATIONS IN GEOTECHNICAL PRACTICES

The following section aims to provide real examples of embodied carbon calculations in geotechnical projects. The Ground Engineering team at Arup Australia Pty Ltd is gratefully acknowledged for providing the case studies presented. Contributors include Sergei Terzaghi, Alvin Chen, Evan Kaillis, Erica Guo, Dongli Zhu, Jeff Clarkeburn, Nick Dewar, and Adrian Callus. Projects details have been left out to maintain the confidentiality of the specific projects.

4.1 FOUNDATION FOR ROAD EMBANKMENT – PRELOADING VS. HEAVY ENGINEERING SOLUTIONS

This case study presents the use of preloaded ground as an alternate design solution to “heavy engineering” for the construction of a road foundation in Sydney metropolitan area (CS1), NSW. Within the site footprint, soft reclaimed and alluvial sediments are present at 12 to 20m depth. The site was previously dredged ground and used for industrial warehouses. Hence, large settlements were expected to occur over a long period of time. Arup was engaged to undertake concept and detailed design for the foundation design of road infrastructure. In contrast to raft footings or piled foundations that have been adopted in neighbouring sites, Arup’s design team investigated the option of preloading and surcharge. Additional field investigations including boreholes, cone penetration tests, dilatometers and laboratory tests were carried out to confirm the ground conditions. Appropriate geotechnical parameters could then be adopted and used in numerical analyses. This led to an alternative design solution being proposed which utilised fill materials and proper drainage techniques (such as PVDs – Prefabricated Vertical Drains) to replace the original tender design of piling and concrete supported slabs.

An embodied carbon assessment was undertaken to understand the carbon savings that occurred from the original tender design (OTD) to the final design. A couple of assumptions have been made throughout the EC calculation; the key items are listed below:

In Both Designs

- Pavement construction activities were not considered in current EC calculation, as similar extent and type of pavement works were adopted, hence, it does not contribute to the difference between baseline scheme and alternate design.
- Minor excavation activities such as landscaping and slope cutting were considered to have minimal impact to the computed EC, compared to the major filling and foundation construction works, thus neglected.
- Landscaping activities have not been taken into account for the calculation of soil volume.
- This distance travelled for the site-won fills was taken as double the length of the site, which was approximately a round trip of 800m.
- CEFs from Stage A1 to A4 were taken from the recommended values as given in Section 3.
- EC_{A5} was calculated based on the proportion of EC_{A5} against overall $EC_{A1\text{-}A5}$, which was assumed to be near constant within each particular type of construction work and was obtained from past project experience from Arup’s in-house database (Arup, 2021), the estimated forecast percentage of $EC_{A5}/EC_{A1\text{-}A5}$ is shown in Table 9. Contribution of EC_{A5} is relatively small compared to the overall EC for usual geotechnical works, as supported by the EFFC example sheet of carbon calculator (EFFC, 2022). This aligned with the assumption of $EC_{A5}/EC_{A1\text{-}A5}$ ratio, as given below.

Table 9: Forecast Ratio of EC_{A5} to overall $EC_{A1\text{-}A5}$ with respective to each work type

Construction Work Type	Work Description - assumed workflow as per Carbon Insights Platform (Arup, 2021)	Estimated $EC_{A5}/EC_{A1\text{-}A5}$ [1]
Earthworks (filling)	Earthwork Fill: assuming the use of an excavator (Cat325), a dozer (Cat D7), an ADT (Bell 30), and a compactor (Cat CS458)	14%
Piling	Rotary bored pile: considering installation of a single pile of 600mm diameter and 20m in length. Assume cast in-situ rotary bored pile, 1% steel reinforcement by volume, pile cap constructed on top of pile, and removal of soil arising from site.	8%
Construction of slab and beam	Raft foundation: unpiled, 1.5m thick reinforced concrete raft, 2% steel reinforcement, 75mm plain concrete binding layer. Soil excavation, disposal, and backfill activities are not included.	3%
Note: 1. Sourced from Arup’s in-house database, Carbon Insights Platform (Arup, 2021)		

For Baseline OTD

- Baseline OTD comprised of construction works of a reinforced concrete slab, beams, and piles, and backfilling works up to the design level as specified in Design Drawings.
- A recycling rate of 85% as suggested by ACT Transport (Transport Canberra & City Services, ACT Government, 2018) and density of 7850 kg/m^3 were taken for steel materials.
- 32MPa concrete with 30% Granulated Blast Furnace Slag (GGBFS) and a density of 2400 kg/m^3 was assumed.
- Site-won materials (or denoted as reused/recycled fills) were used for backfilling. This assumes the lowest bound of embodied carbon emission for comparison.
- Steel reinforcement in concrete slabs and beams was taken as 1%, and 2% in piles.
- Installed piles were terminated at bedrock level.
- Site activity emissions for piling and construction of concrete structures (Stage A5a) has not been captured in EC_{A5} embodied carbon calculations due to the limitations in direct measures of fuel and energy consumption for these activities, especially for those involving a large proportion of labour work such as formwork and concreting (as discussed in Section 3.3).

For Final Design

- Final design comprised of filling works in the form of preload, surcharge, and imported/reused fills.
- A thin layer of drainage materials was assumed to consist of gravel fillings (approximately 100mm thick).
- The volume of fill did not consider the effect of settlement, it was computed only from the existing level to the proposed design level plus surcharge level, if required.
- Imported fills were utilised for preload and surcharge fill during the project. Site-won spoil was available from piling of the building structure on site, however, was deemed unsuitable and unable to be reused due to having excessive moisture content as a result from prolonged exposure to inclement weather.
- Reused/recycled/site-won fills was also calculated as a sensitivity check. The true project EC for the final design, which partially utilised reused/recycled soil, would be within the range of the imported and reused fill results.

A summary of the embodied carbon results is given below (Table 10). Figure 4 presents a comparison between baseline OTD and final design. Figure 5 to Figure 7 shows the computed EC at each stage within the same design option. A few key findings from the EC results study are listed below:

1. The final design (with imported fills) equated to 30% or less of the total embodied carbon for baseline OTD solution (lowest bound of EC considered in OTD).
2. The use of site-won materials (reused fills) imposed ~50% reduction in the design scheme, when comparing Final Design option 1 (imported fills) and option 2 (reused fills).
3. The materials production factor contributed to more than 90% of the total embodied carbon in the construction of concrete structure, however, significantly reduced to 70-80% when earthworks (i.e., ground improvement solution) replaced rigid engineering foundations.

Table 10: Results of embodied carbon emissions for design options of CS1

Baseline Scheme (OTD)		**Final Design (Option 1 – Imported fills)**		**Final Design (Option 2 – Reused fills)**	
comprises of construction works for slab, beams and piles		*comprises of ground improvement works including preload and surcharge*			
Stage	**EC (*kg.CO₂e*)**	**Stage**	**EC (*kg.CO₂e*)**	**Stage**	**EC (*kg.CO₂e*)**
A1-A3	3,385,859	**A1-A3**	744,880	**A1-A3**	382,341
A4	100,483	**A4**	150,399	**A4**	31,672
A5	148,119	**A5**	145,743	**A5**	67,397
Sum	3,634,461	**Sum**	1,041,022	**Sum**	481,410
-		$\frac{EC_{A1-A5,FD}}{EC_{A1-A5,OTD}}$	29%	$\frac{EC_{A1-A5,FD}}{EC_{A1-A5,OTD}}$	13%

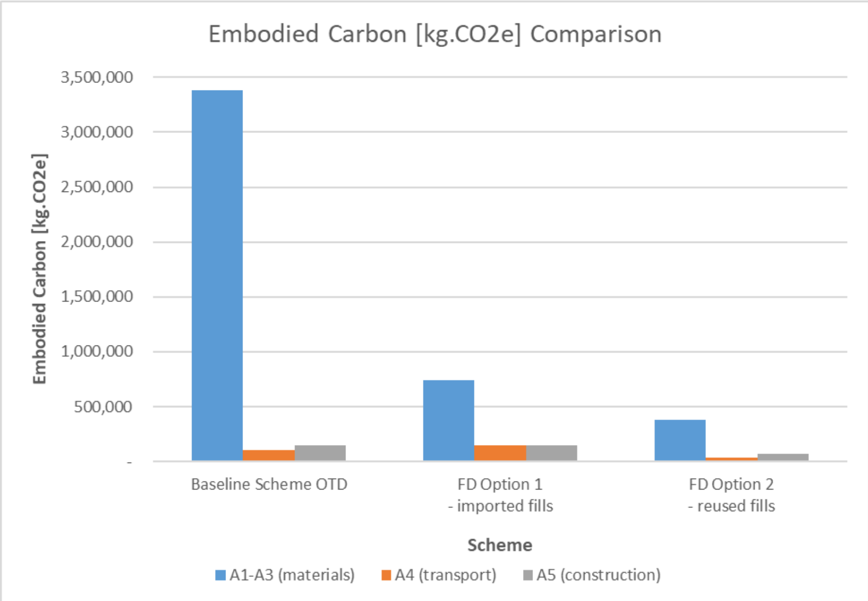

Figure 4: Embodied carbon comparisons between OTD and final design

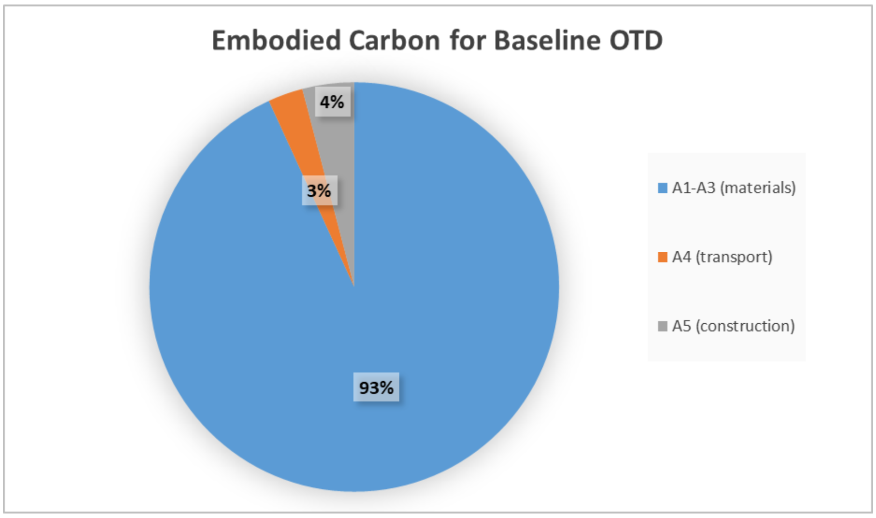

Figure 5: Embodied carbon distributions for OTD

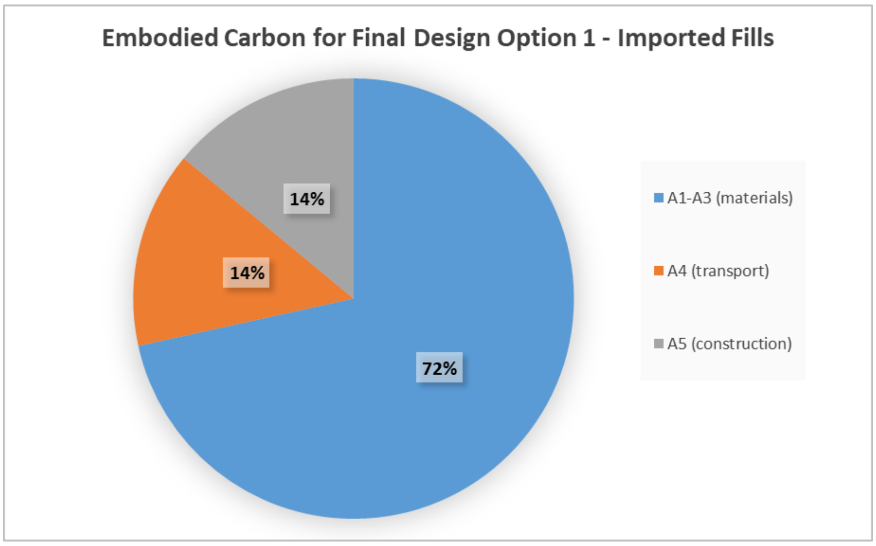

Figure 6: Embodied carbon distributions for FD option 1 – imported fills

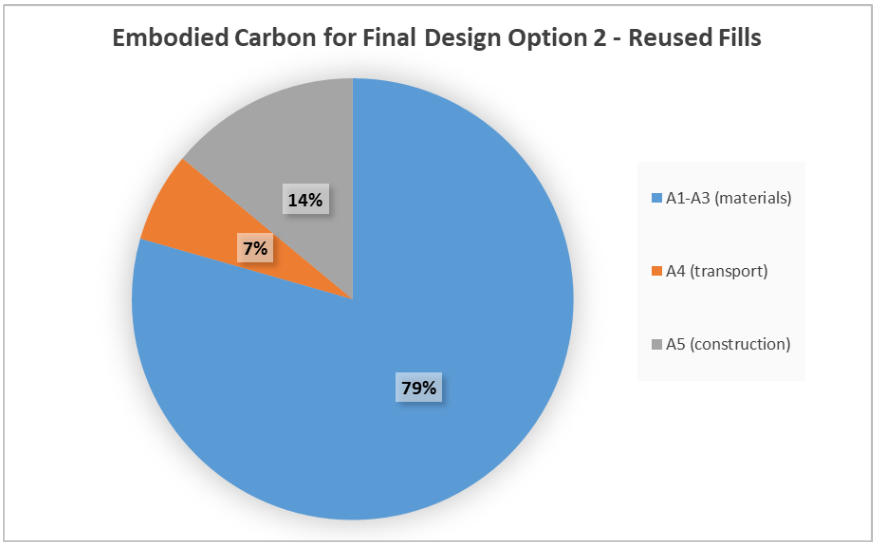

Figure 7: Embodied carbon distributions for FD option 2 – reused fills

4.2 UTILITIES INFRASTRUCTURE – TRENCH EXCAVATION

This case study focuses on a utility infrastructure project located in the Greater Western Sydney region. As part of the concept design, a geotechnical desktop study was carried out to inform the key constraints and opportunities for the design. This included a simple embodied carbon calculation comparing the two most common methods of open trench excavation: battered excavation of 1V:2H slope and a supported shoring trench. The following assumptions have been made to facilitate the EC calculations:

- The major activity involved in the battered option is excavation; the major activity involved in the shoring option is concreting plus excavation. The work of excavation includes both soil and rock cutting.
- EC per unit meter of trenching was evaluated for optioneering purpose, instead of a full EC study. This was considered more useful as the detailed design inputs are often subject to change (e.g. alignment).
- A general trench depth of 7m was adopted for excavation extent (with 1m width), assumed 4m depth of soil overlaying the rock stratum. Slope gradient of 1V:1H in rock, and 1V:2H in soil were assumed, sample section details as shown in Figure 8.

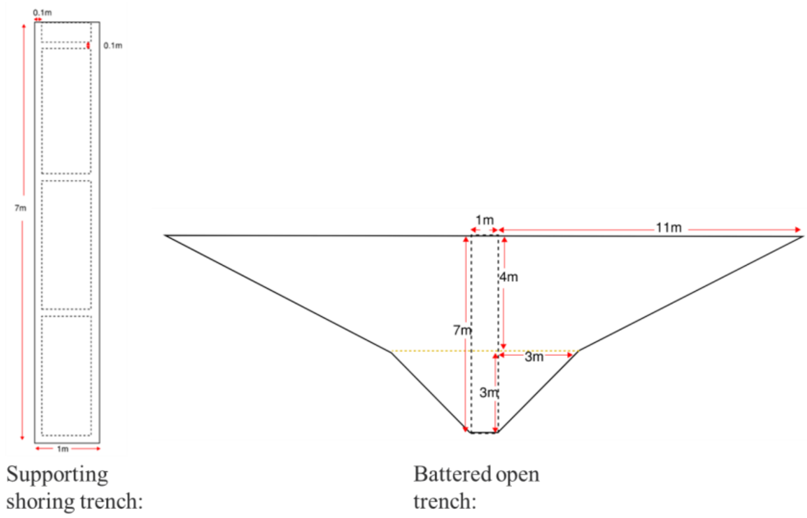

Figure 8: Trench cross-section showing possible excavation options for EC evaluation

- Unit mass of 20 kN/m^3 and 24 kN/m^3 were adopted for soil and rock respectively.
- $CEF_{materials}$ and $CEF_{transport}$ were taken from the recommended values, as given in Section 3.
- No engineering materials were created/produced due to battered open trench option, hence, $EC_{A1\text{-}A3} = 0$ for the battered open trench.
- Road transport (on land) with average laden was assumed for excavated materials delivery.

- A total distance of material transportation was taken as 25*km*, based on site constraints.
- For EC_{A5} calculations, only the embodied carbon emissions that contribute to the machinery activities were considered given that they were measurable. As the excavation activity being the dominant type of work, an excavator with an assumed working performance of 20 *L/hr* and 75 m^3/hr of fill removal was adopted, based on site practices and product catalogues.
- Only EC_{fuel} was considered here as part of EC_{A5} due to the constraints of insufficient research and data in collating the amount of electricity used on site, hence, the component of $EC_{electricity}$ was omitted in this simple exercise (recalling $EC_{A5} = EC_{fuel} + EC_{electricity}$ from Section 2).
- The CEF_{fuel} for an excavator was taken as the carbon emission factor for unit diesel used (i.e., 2.7 $kg.CO_2e/L$), with reference to the open source from the National Transport Commission (NTC, 2019).
- For the supported shoring trench, a minimum steel reinforcement of 0.8% was assumed, without considering recycling.
- Based on experience and usual practices, time required to install a single shoring box and each battered open trench sectional length (per box) was assumed to be 0.5*hr*/section and 6*m* respectively.

The results from the EC assessment (per meter alignment) are summarised below (Table 11), the battered trench option was found to outperform the shoring box option by approximately 20%. However, this assumes that the shoring structure is built along the whole chainage. Reusing the shoring box reduces the mass of engineering materials (i.e., EC_{A1-A3}), which contributes to nearly 90% of the calculated EC. Hence, it is suggested that the option of reusing these shoring boxes should be considered in practical application, noting that this is dependent on particular project constraints. Figure 9 to Figure 11 illustrated the computed EC of the project options, as well as their distributions per different stages.

Table 11: Results of embodied carbon emissions for trench excavation options

Stage	**Embodied carbon for battered open excavation ($kg.CO_2e/m$)**	**Embodied carbon for supported shoring trench ($kg.CO_2e/m$)**
A1-A3	0	900
A4	808	105
A5	16	10
Total	**824**	**1014**

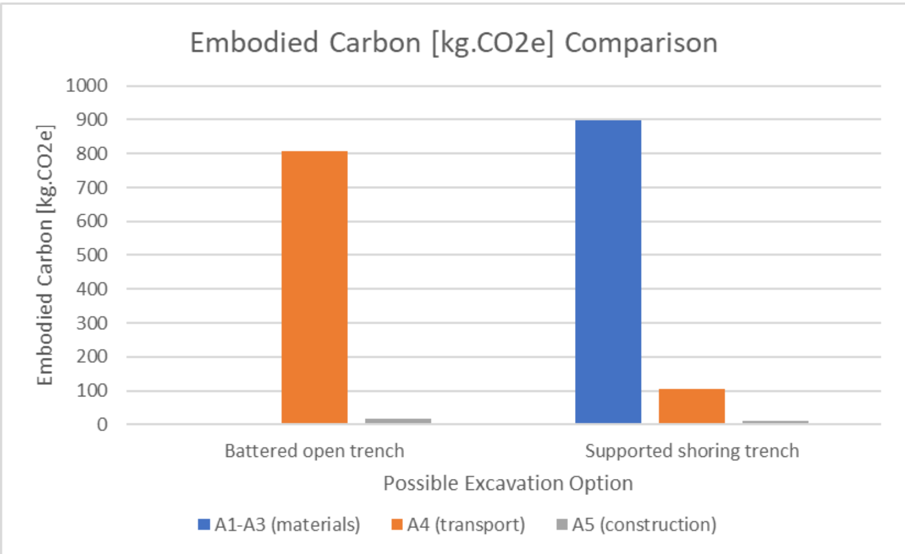

Figure 9: Embodied carbon comparisons between two trench excavation options

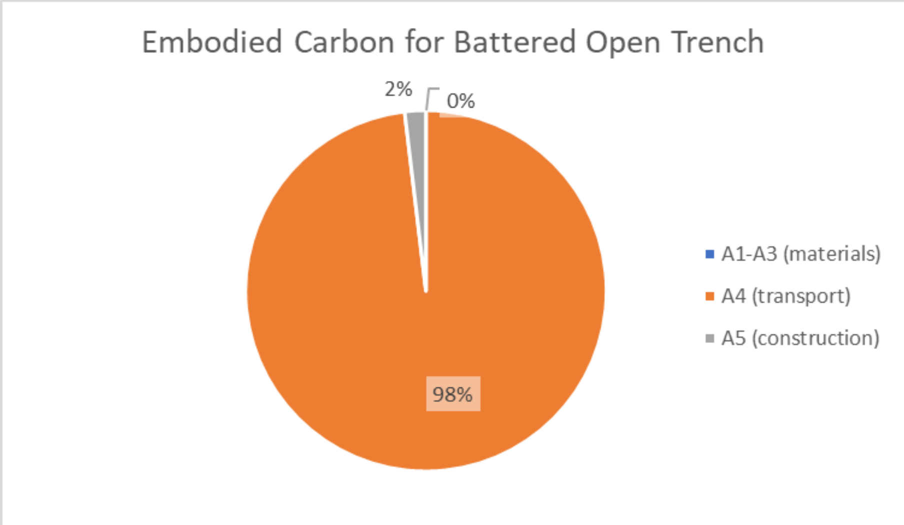

Figure 10: Embodied carbon distributions for battered open trench excavations

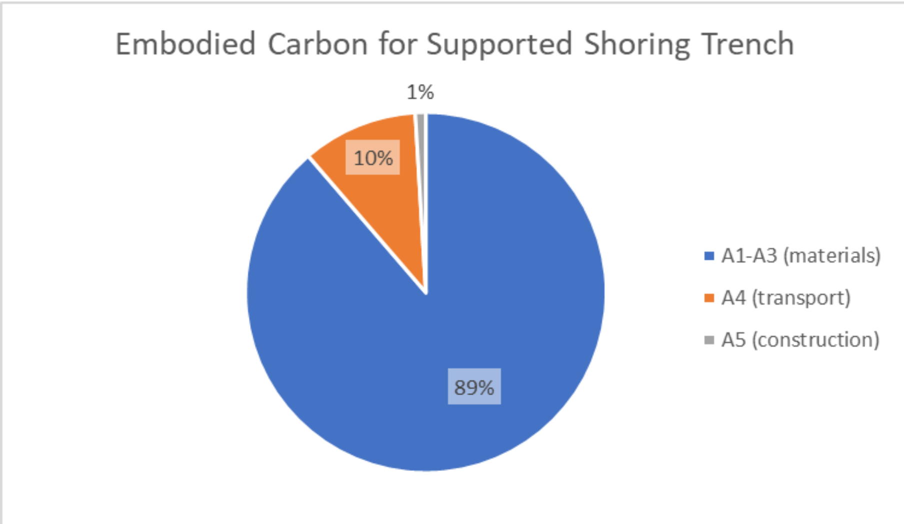

Figure 11: Embodied carbon distributions for supported shoring trench excavations

4.3 PERMANENT TUNNEL LINING – TENDER DESIGN

As part of a recent tunnel tender design, the embodied carbon of four stations was calculated. The assessment allowed the designer and client to understand the 'carbon heavy' components of the design and allowed further optimisation. The following key assumptions and limitations were made as part of the embodied carbon assessment:

- Computed embodied carbon was measured in tonne of carbon dioxide emissions (*t.CO2e*).
- Only Stage A1-A3, and A4 were considered. Stage A5 EC calculation was excluded due to the high level of engineering study required and data inadequacy at this early phase of project consultation.
- For the purpose of simplicity, the carbon factor for PL2 – C40/50 was applied to all concrete in the tunnel station design. Whilst the vast majority of concrete grade/mix was PL2 – C40/50, there was also minor amounts of PL4 – C50, and PL3 – C50, CEM III/B.
- The volume of steel within the concrete varied between approximately 0 to 6% depending on the structural component.
- Reinforcing steel assumed to be virgin (i.e., contain 0% recycled content).
- The embodied carbon from rock bolts and waterproofing components were ignored for simplicity.
- Average laden was assumed for road transport emissions in Stage A4 calculations.
- Spoil removal was captured in Stage A4 instead of Stage A5.
- Disposal of spoil did not consider the bulking factor of soil.

The results of the EC assessment are provided below (Figure 13 to Figure 15). The majority of EC is from the material source (Stage A1-A3) rather than the transportation (Stage A4). Figure 15 showed that the majority of EC_{A4} was related to spoil removal, which is expected for a tunnel project type. However, this result should prompt the designer to consider recycling the spoil (with proper treatment if required). This could surpass the efficiency of any decarbonisation technique on concrete or steel when it comes to the consideration of EC_{A4}. The carbon "credits" for recycling the spoil are typically considered in Stage D, which is outside the system boundaries of this simple A1-A4 calculation.

Results from Figure 14 show that even though steel contributed to only a small amount of volume/mass (generally less than 6% of the concrete volume), their computed $EC_{materials,A1-A3}$ are comparable to each other. In other words, the reduction of steel consumption, or use of recycled steel may be more advantageous in reducing carbon emissions than reducing the use of concrete at the same scale.

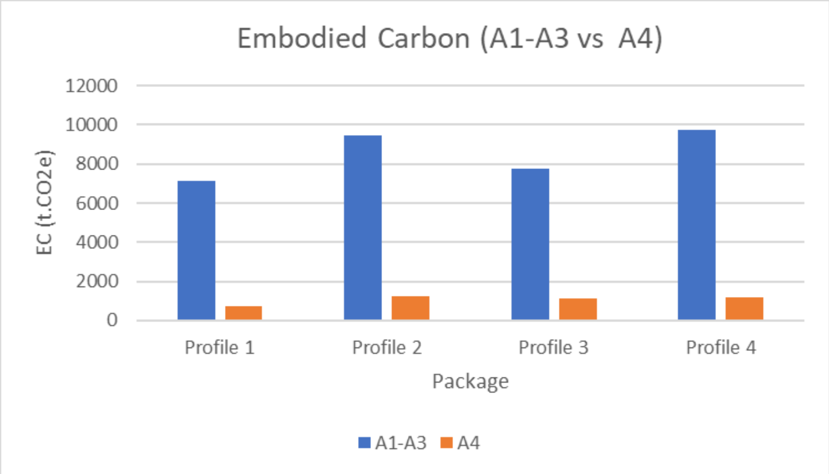

Figure 12: Embodied carbon distribution of Stage A1-A3 and Stage A4

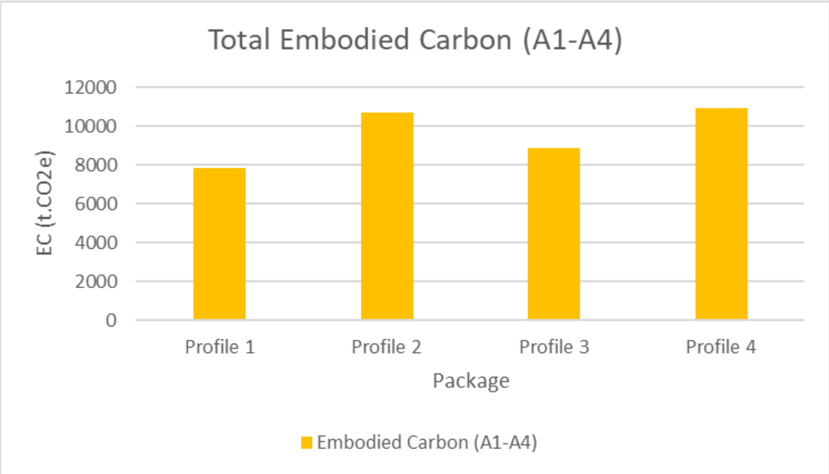

Figure 13: Total embodied carbon for Stage A1-A4.

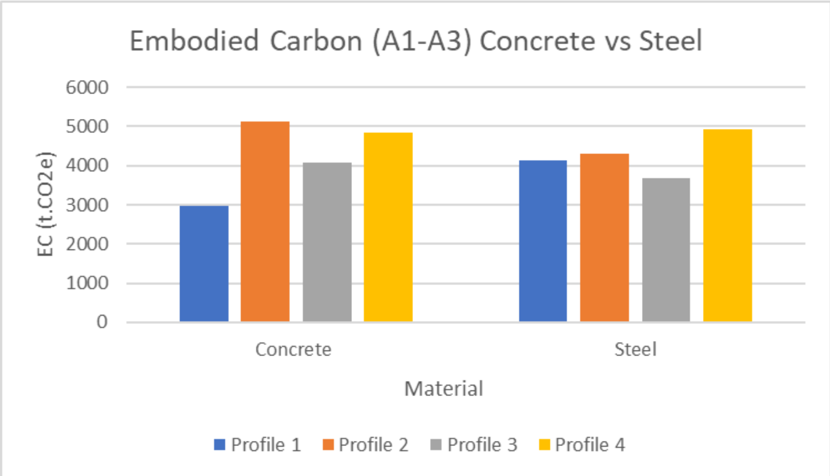

Figure 14: Embodied carbon distribution of materials Stage A1-A3.

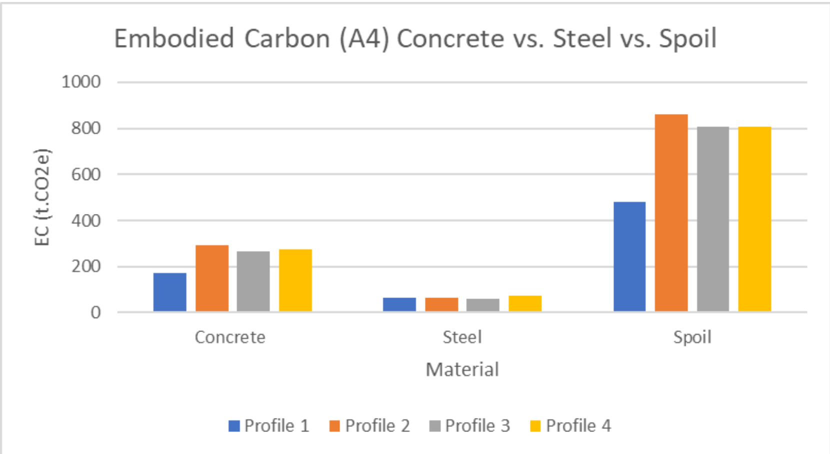

Figure 15: Embodied carbon distribution of materials at Stage A4 including soil removal

5 METHODS OF DECARBONISATION

Sustainable development is defined as the growth of human living standards that ‘meet the needs of present without compromising the ability of future generations to meet their own needs’ (Brundtland, 1987). Fundamentally, it would obligate to satisfy the three main pillars of environmental, social, and economic objectives. The methods discussed here are targeted to tackle the industrial practices of reducing carbon emissions from a viewpoint of geotechnics and to fulfill the three main perspectives of sustainability goals.

The following waste hierarchy is introduced in Figure 16. It showcases the priority for the most efficient ways to minimise waste and energy consumption, hence, carbon emissions. The subsequent chapters should be read according to the order of this ‘reverse pyramid’, with the most preferable methods for decarbonisation at the top, through to the least at the bottom of the pyramid.

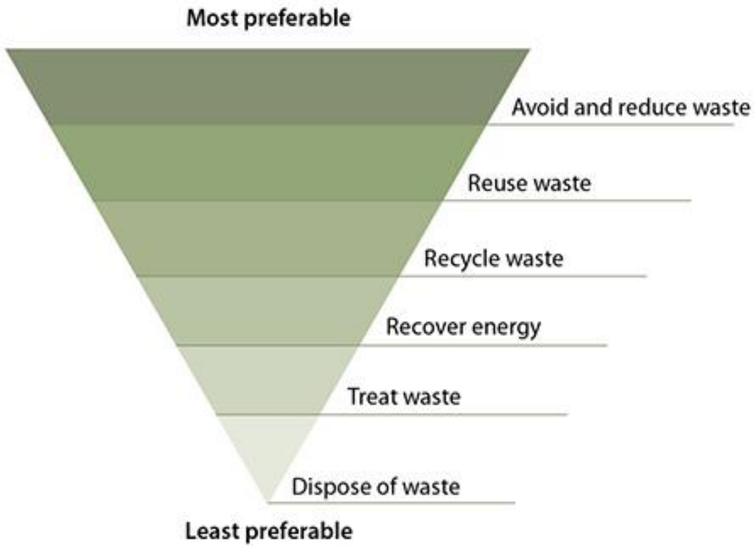

Figure 16: Waste hierarchy ‘reversed pyramid’ (NSW EPA, 2022)

5.1 LEAN DESIGN APPROACH

The need of establishing a ‘lean design approach’ has escalated in recent times. It is typically the most preferable choice to ‘avoid and reduce’ carbon emission. The principles of lean design were evolved by Womack and Jones in 1996 that consolidated the concept into the following (Womack & Jones, 1996):

1. Identify value – client’s needs, project scope and target function of the engineering asset.
2. Map and create the ‘work-flow diagram’ – to ensure the allocated resources and work tasks are essential to the target goals and are constructible.
3. Establish pull – deliver the product/option scheme in a ‘work-on-demand’ basis that avoids excessive resource usage and/or overdesign. For instance, it could be achieved by a more calibrated estimates of Building Quantities to reduce inaccuracy assigned. These calculations may not need to be more detailed but should be calibrated with data collected from as-built material volumes and comparing with estimated design volumes to establish design to reality ratios. Similarly, 3D modelling and consideration of the appropriate level of detail at a given project stage may be used to more accurately estimate volumes from an early project stage.
4. Seek perfection – repeat step 1-3 until significant drop-in work delivery time and high efficiency in resource usage could be achieved.

Implementing the above lean design thinking into geotechnical practice, engineers should undertake appropriate optioneering assessments to outline the project aims and create designs that make best use of materials, whilst limiting embodied carbon expended in transport and construction. Considerations could include: the use of project specific ground investigations to optimise design parameters; installation of instrumentation and monitoring to verify design predictions with real-life observations, following the “observational method” (as stipulated in Eurocode 7) where designs are monitored on site and engineering solutions undertaken on a per need basis, and considering ground improvements as an alternative to ‘hard engineering’ solutions. This was clearly demonstrated in the CS1 study, presented in Section 4.1. The choice of using ground improvement methods (surcharge and preloading) allowed the minimization or even elimination of concrete and steel utilisation. This substantially reduced the amount of waste generated and associated carbon emissions. Sufficient ground investigation data in early design stage also enabled appropriate design parameters, justification of reduced partial factors and overall, a more efficient design. Furthermore, settlement monitoring provided understanding of the time required and degree of surcharge and preloading required for the design.

5.2 MATERIAL SELECTION

Better choice of engineering materials also helps to 'reduce', 'reuse' or 'recycle' resources. As shown in Section 3.1, the $CEF_{materials}$ vary for different types of concrete and steel. Therefore, it is imperative that the designer specifies the appropriate material for construction that meets both; the minimum design standards (avoiding overdesign i.e., lean design approach) and selects the lower carbon material where possible.

Examples of this include: The use of different cement mixes that contain lower embodied carbon additives (i.e. GGBS Ground Granulated Blast Furnace Slag or fly ash) to replace traditional (carbon 'heavy') cement clinker. Other examples include using alkali-activated materials to substitute Portland Cement and replace steel with micro-macro synthetic (EFFC, 2022).

Using steel products manufactured with higher recycling rate or recycled fills/aggregates instead of imported fills could also help to reduce carbon footprint. EcoSheetPile and EcoSheetPile Plus from ArcelorMittal is an example of better material selection. They benefit from a manufactured recycling scrap rate of 100%, and the use of 100% renewable energy to power the electric arc furnace as part of steel production (ArcelorMittal, 2023). Other local examples of 'green' manufacturing is the implementation of renewable hydrogen electrolyser in blast furnaces by BlueScope Steel (Peacock, 2022). The 'green steel' industry is rapidly growing across the industry and should be more widely promoted by engineers and developers to strategically include the use of recycled or 'green' products as part of the contractual terms. The choice of local suppliers and yards also minimises the transport distance of materials to and from the project site, which may play an important role when EC_{A4} is crucial to the project overall EC (for instance, in the example of battered open trench excavation option given in Section 4.2).

5.3 REDUCTION OF WASTE DISPOSAL

As illustrated in the example of Tunnel Tender design (in Section 4.3), the contribution of soil removal (either EC_{A4} or EC_{A5}) can be a significant contribution to the overall embodied carbon assessment. While waste disposal can often be reduced, it cannot always be removed entirely. An example of reusing site-won materials is provided in Section 4.1 (CS1). With reference to the waste hierarchy diagram, this would elevate the mitigation method from the bottom (disposal of waste) to 'reuse'. Examples and case studies of waste disposal include:

- The case study discussed in Section 4.1, showed an overall EC reduction of nearly 50% could be achieved by reusing soil aggregates as fills. This outcome emphasised the significance of waste reuse, especially in the context of earthworks. Another study to be considered, reviews the reuse of tyre bales as lightweight fill (Kidd, et al., 2009).
- Treating spoil is another potential way to significantly reduce waste disposal, as outlined in the Tender Design case study (Section 4.3). Whilst the treated soil may not be able to be reused on the project it was sourced from, it could be used as filling works on other projects. The industry is becoming increasing more aware of the opportunity of sharing resources or wastes to minimize soil resources being sent to landfill.
- Reusing temporary work structures could also help to save excessive carbon emissions. The case study (Section 4.2) shows how a reusable (pre-cast) shoring structure can significantly reduce the quantities of material production along the alignment if the structural performance of the temporary frame has been certified and confirmed by the field engineer.

  Soil stabilisation is another geotechnical area that could make use of waste-transformation. Examples of reused products that have been used to aid in soil stabilisation include ground granulated blast furnace slag, furnace bottom ash, and pulverised fuel ash that could be reused in a concrete mix (Pantelidou, et al., 2012).

6 CONCLUSIONS AND RECOMMENDATIONS

The methodology and simplified formulae for embodied carbon (EC) assessments is discussed in this paper, consisting of material production (A1-A3), transport of materials (A4) and construction (A5) stages. A quick guidance on the selection of carbon emissions factors with respect to the most commonly adopted engineering material types is presented. The application of these calculations was given in Section 4, which took the examples from real projects and demonstrated the significance of having the EC assessment done in early design stages to ensure best EC reduction strategies were implemented. Based on the example projects, the key findings on performing EC evaluations were summarized as follows:

- An earthworks foundation typically contains lower embodied carbon than a traditional 'hard' engineering structure (such as the reinforced concrete piling foundation in CS1, Section 4.1 and shoring design in trench excavations, Section 4.2).
- The use of reused or recycled aggregates as fill materials could impose a large reduction in EC (nearly ~50% in the presented case study). Therefore, best practice would be to always consider if on-site won materials are suitable for re-use or filling purposes (Section 4.1).
- Similarly, reducing the travel distance of soil or rock from spoil removal activities may play an important role in tunnelling projects (as per Section 4.3). This could be done by reusing the excavated materials by means of proper treatment methods, acquiring a local manufacturer for materials production, and local receiver as waste disposal.
- Although a ‘hard’ engineering structure (such as a reinforced concrete slab) typically has a higher EC footprint than an equivalent earthwork activity. If the ‘hard’ engineering structure could be reused, the EC may be significantly reduced. For example, the trench excavation case study (Section 4.2) revealed that if the contractor was able to reuse shoring boxes along the alignment, this could reduce the $EC_{A1\text{-}A3}$ by up to 90%.
- Early involvement of EC calculation in tender design stage (Section 4.3) helps identify the key areas of concern in later development stages. This can assist in decision making for site selection and target the focus of implementing carbon reduction strategies.
- Overuse of steel reinforcement (as per finding in Section 4.3) should be avoided where possible, or ‘green steel’ and steel manufactured with higher recycling rate should be adopted by the industry.

Some decarbonisation methods have also been introduced, which should be prioritised in the order of ‘reduce’, ‘reuse’, ‘recycle’, and ‘disposal’. Three main approaches have been addressed, they include lean design thinking, material selection, and reduction of waste disposal. The value of EC calculations has also been reinstated such that it enables the designer to understand the carbon ‘heavy’ components of the design and allow further refinement for decarbonisation as per the strategies listed. More research and further investigations are recommended to better quantify the embodied carbon calculation of Stage A5 (construction). It is recommended that either alternate formulation of EC_{A5} should be developed based on more measurable quantities in early design stages or collation of more extensive databases for CEF_{A5}. Therefore, the overall EC and global warming potential of each individual engineering option could be more easily and accurately assessed to inform key decision making.

7 ACKNOWLEDGEMENTS

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MODIFIED ASPHALT BY COFFEE CUP FIBRES: AN OPTIMUM MIX DESIGN USING RESPONSE SURFACE METHOD

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ABSTRACT

Stone Mastic Asphalt uses fibres to stabilise high binder contents. The fibres are typically made from natural cellulose, and prevent binder draindown. This study investigates the use of post-consumer coffee cups (liquid paperboard) as a recycled fibre alternative. Coffee cups (CC) use laminated polyethylene as a liquid barrier, making them challenging to recycle in existing paper and plastic recycling systems; approximately one billion coffee cups are landfilled annually in Australia. In this research, coffee cup fibres are used up to 0.6% by total weight of the asphalt mixture across fourteen mix designs. The mixes are optimised by the response surface methodology using the Design-Expert software. Models are created based on Marshall and draindown test results. This study finds that shredded coffee cups effectively reduce the draindown, reduce flow of the asphalt mixture, increase stability, and increase the air voids. The LDPE content slightly increased the viscosity and the softening point of the asphalt binder and dropped its penetration value. Overall, the asphalt mixture containing 0.4% coffee cups (by total weight of the mixture) met all specifications in Australian Standards and performed similarly to the commercial cellulose fibre. This paper also provides practical insights from field trials conducted in Western Sydney using the optimised parameters identified. The pavements will continue to be monitored to develop long term performance comparison between CC and traditional fibres in SMA applications.

1. INTRODUCTION

Fibres have been used to modify asphalt mixtures for many years [1], and a range of their length, width, diameter, and material compositions are applied [2]. The common materials that are used to produce fibres for asphalt modification are carbon [3], glass [4], basalt [5], polyethylene terephthalate (PET) [6], nylon [3], polypropylene fibre (PP) [7], metal [8], and cellulose [9]. Generally, fibres are used to modify the viscoelasticity of the asphalt mixture [10], increase the dynamic modulus [11], improve creep and rutting resistance, and enhance the fatigue life of the mixture [12]. In addition, fine fibres, such as cellulose fibre, are usually used to prevent the draindown in asphalt mixtures with high binder content such as SMA [12,13].

Cellulose fibres are mainly composed of cellulose, hemicellulose, and lignin from natural wood [14,15]. Wood chips are pulped to manufacture paper, cardboard and many other products, and represent 6% of total global energy consumption [16, 17,18] Annually, 16 billion coffee cups are used worldwide, which leads to the logging of 6.5 million trees [19]. In Australia, around one billion coffee cups are landfilled each year [20]. This is due to the hard-to-recycle nature of the cups [21]. Paper cups need to hold liquid, and to do so; they are lined with a plastic layer as a moisture barrier [22].

The lining is typically low-density polyethylene (LDPE), and needs to be separated prior to processing through existing recycling systems; this separation process is expensive and complex [22]. Since the decomposition of plastic-lined coffee cups takes around thirty years [23], a domestic solution to recycle liquid paperboard is indispensable. In this study, we examine the application of the post-consumer waste coffee cups, as an alternative source for cellulose fibre in stone mastic asphalt.

The utilisation of waste fibres in stone mastic asphalt and open-graded asphalt has been investigated before [24,25], and constructive impacts are reported. For instance, It has been revealed that carpet fibres can increase resistance against moisture damage [24], old corrugated paper and recycled magazines can improve rutting resistance and decrease binder draindown [26], and recycled textile fibre can improve the fatigue life of the mixture [27]. Composite fibres and liquid paperboard have not been previously investigated in this application.

This study has been conducted in cooperation with State Asphalt NSW and Closed Loop Environmental Solutions [28], and assesses the potential for this waste as a replacement for commercial fibres in SMA[29].This research benchmarks the properties of SMA mixes with CC, against a commercially available fibre (JRS Viatop).

Table 1: Abbreviations

Abbreviation	Definition	Abbreviation	Definition
CC	Coffee Cups	PET	Polyethylene Terephthalate
HV	High Viscosity Binder	PP	Polypropylene
LDPE	Low Density Polyethylene	RSM	Response Surface Methodology
LPB	Liquid Paperboard	SMA	Stone Mastic Asphalt
MFR	Melt Flow Rate	TfNSW	Transport for New South Wales

2. MATERIALS AND METHODS

2.1. MATERIALS

Bitumen C320 and C450, typical bitumen used in Australia, were used in this study [30]. These bitumen grades were obtained by mixing bitumen C170 and high viscosity (HV) bitumen as instructed by the supplier, Viva Energy Australia Ltd. The aspects of bitumen C170 and HV are shown in Table 2. To obtain bitumen C320, 56% C170 was mixed with 44% HV, and to obtain bitumen C450, 40% C170 was mixed with 60% HV.

SMA10 using C450 binder was used as a control group for Marshall specimens in this study. C320 was used for bitumen tests to investigate the sensitivity of dry-mixed binder modification from the plastic fraction in CC. C450 has a viscosity of 1000pa.s at 60°C [31] and is less prone to viscosity change than C320 (with a viscosity of around 300 at 60°C) [32].

Table 2: Bitumen properties

Specification	Test	HV	C170	Units	Limit
AS 2341.7	Density @ 15°C	1042.6	1040.3	kg/m^3	
AS 2341.2	Ductility @15°C after RTFO[1]	15	-	Pa. s	140 - 200
	Dynamic Viscosity @ 60°C	763	171	Pa. s	
	Dynamic Viscosity @ 60°C after RTFO	2290	362	Pa. s	
	Dynamic Viscosity @ 60°C - as % of original after RTFO	301	212	%	
AS 2341.3	Dynamic Viscosity @ 135°C	0.717	0.37	Pa. s	0.25 - 0.45
AS 2341.14	Flash Point - Frequency	>300	>300	°C	Min 250
AS 2341.8	Matter Insoluble in Toluene - Frequency	<0.1	<0.1	mass %	Max 1.0
AS 2341.12	Penetration @ 25°C/100g/5s	33	72	0.1 mm	Min 62

1- RTFO stands for rolling thin-film oven.

Aggregates (coarse and fine) and mineral fillers (baghouse dust and hydrated lime) were supplied by State Asphalts NSW. The SMA10 mix design conforms to TfNSW R121 – Stone Mastic Asphalt specification particle size distribution requirements [33].

The coffee cups were supplied by Closed Loop Environment Solutions from post-consumer collection sources through Simply Cups. The coffee cups are shredded to 5mm minus and estimated to contain 8-10% LDPE and > 92% paperboard by weight from State Asphalts NSW quality audits. A comparable LDPE film grade (LD2426K) was identified and supplied by Primaplas Pty Ltd from GC Marketing with [MFR (g/10min), Density (g/cm^3), Melting point (°C)] = [21, 0.91, 106].

The reference fibre is Viatop supplied by J.Rettenmaire and Sohne [34] for making a control sample. Viatop is a pelletised blend of bitumen and cellulose fibre which is added to the asphalt as a binder drainage stabiliser.

2.2. PREPARATION OF BINDER SAMPLES

The base C320 binder was modified with 0%, 0.5%, and 1% LDPE to provide a characteristic reference depending on the level of binder dissolution and dispersion from the CC, assuming zero binder dispersion, to complete binder dispersion. At a maximum of 0.6% CC (by total weight of the mixture), and 6-7% binder content, the estimated LDPE weight fraction in binder is between 0.4%-0.9%.

For the binder preparation, 700 g of C320 binder was heated 180 °C in an oven, then placed on a 180 °C thermostatically controlled hot plate. The binder was gradually blended with LDPE using a Silverson LM5-A Shear Mixer at 3000 rpm for 60 minutes to achieve a homogeneous blend.

The modified binder samples were tested for their softening point, penetration, and viscosity at 60, 135, and 165 °C and the results were compared with the properties of virgin bitumen C320 and C450.

2.3. PREPARATION OF ASPHALT MIXTURES

Aggregates and bitumen were preheated two hours at 160 ± 5 °C. The hot aggregates were added to a temperature-controlled Bitumen Asphalt Mixer (160°C) with ambient temperature fibres and mixed for 60 seconds. The hot bitumen was introduced into the asphalt mixture and mixed for 10 ± 1 minutes before placement into a Marshall specimen mould. Specimens were compacted by 50 blows of the hammer at the temperature of around 150 ±3 °C, as specified in AS 2891 [35].

2.4. CONVENTIONAL TESTS OF BITUMEN

Given low polymer modification rates, the 0%, 0.5%, and 1.0% LDPE modified bitumen was evaluated against Australian viscosity-grade tests; penetration at 25 °C, viscosity at 60 and 135 °C, penetration at 25 °C after short-term ageing, and viscosity at 60 °C after short-term ageing. The penetration test indicates the stiffness or consistency of an asphalt binder [36]. Penetration testing was conducting in accordance with AS 2341.12. The viscosity of an asphalt binder is an indication of its resistance to flowing [37,38].

The addition of plastics to bitumen, regardless of its type, increases the viscosity [39]. The viscosity test was conducted at three temperatures: 60, 135, and 165 °C, and the viscosity at 60 °C was measured by a vacuum capillary viscometer according to AS 2341.2 using a PSL-100 tube. The viscosity of the samples at 135 and 165 °C was measured by a Brookfield viscometer following AS 2341.4.

Short-term ageing samples were conditioned using a rolling thin film oven (RTFO) for 85 minutes at 163 °C according to ASTM 2872.

2.5. MARSHALL AND DRAINDOWN TESTS

As outlined in ASTM 6927 [40], a minimum of three Marshall specimens, for each mix design, must be tested. In this study, four Marshall specimens for a given mix design were prepared, and the average of the test results (Stability, Flow number, AV, etc) was considered representative of the mix [40]. Binder content and CC content were optimised across binder content ranges from 6-7% by mass and CC content ranges from 0 to 0.6% by mass of the asphalt mixture respectively.

Although AASHTO M 325 recommends a maximum stabiliser content of up to 0.4% of the total weight of the mixture [42], this study evaluates the inclusion of up to 0.6% coffee cups to attain a better understanding of their impacts. Fourteen different mix designs, and three samples for each mix design (a total number of 42 specimens), are considered. An SMA mixture with 0.3% Viatop by total mass of the mixture, as recommended by the manufacturer, was also prepared to compare commercial fibres and the coffee cups.

Specimens were cooled to room temperature to determine their bulk specific gravity (Gb) [43], theoretical maximum specific gravity (Gt) [44], air voids (AV) [45–47], and thickness [48]. The specimens were conditioned in the water bath for 30-40 min at the temperature of 60 °C before Marshall stability, and the flow tests were conducted [40]. The measured stability values were corrected based on the volume and thickness of the specimens according to ASTM 6927 [40]. A draindown test was also conducted for each mix design according to RMS T648 [49].

2.6. RESPONSE SURFACE METHODOLOGY (RSM)

RSM uses a collection of statistical methods to evaluate the effects of the experimental factors [50]. RSM is used in this study to create a model based on the Marshall and draindown test results to optimise the mix design. The experiments are conducted based on the Central Composite Design (CCD) strategy using the Design-Expert software [51,52]. CCD is a fractional factorial experiment design that provides relationships between responses (dependent variables) and factors (independent variables) at different levels [50]. Given the large number of test permutations and factors evaluated, RSM and CCD provides a significant reduction in experimental cost whilst providing insight into parameter optimisation.

Binder content is the first factor ranging from 6% to 7%, and CC content is the second factor ranging from 0 to 0.6%. Therefore, experiments with 6% or 7% binder content and 0% or 0.6% CC are factorials, experiments with 6.5% binder content and 0.3% CC are centrals, and the rest of the experiments are axials. In this study, six central, four axial, and four factorial points were considered for the CCD strategy (a total of 14 mixes).

Independent variables (factors), defined as (x_1) and (x_2), are binder and CC contents, respectively. For statistical calculations, the real values of a factor (X_i) was converted to a coded value (dimensionless) as (x_i) by Equation 1 [53].

Equation 1:

$$x_i = (X_i - X_0)/\delta X$$

Where X_0 is the real value of a factor at its central level, and δX is the step change value between the low level (-1) and the high level (+1). The dependent variables (responses), which are the indicators of the performance of the mixture, are defined as the (Y) function. The responses are Marshal stability, flow number (FN), Air Voids (AV), and the draindown. To calculate the dependent variables (responses) the polynomial model presented in Equation 1 was introduced [54,55].

Equation 2:

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_{12} X_1 X_2 + \beta_{11} X_1^2 + \beta_{22} X_2^2 + \varepsilon$$

Where Y is the dependent variable and X_i, X_i^2, and $X_{ij}$are the effects, square effects, and interaction effects on the independent variable, respectively. β_i, β_{ij}, and β_{ii} are the coefficients of the linear, interaction, and squared effects. β_0 is the intercept parameter and ε is the error. Table 3 lists the variables and the acceptable ranges according to Australian Standards. These values will be considered as the basic criteria to optimise the mixture containing CC. To justify the adequacy and significance of the predicted model, analysis of variation (ANOVA) and (P-Value>0.05) significance were used [56,57,58, 59].

Table 3: Dependent variables and the acceptable ranges based on Australian specifications.

Response	Name	Units	Acceptable range	Desirable value	Reference
R1	Stability (corrected)	kN	> 5.5	Peak point	[60]
R2	Flow Number	mm	1.5 - 4	2 - 3.5	[60]
R3	Air Voids (AV)	%	4 - 6	4 - 6	[60]
R4	Bulk specific gravity (Gb)	g/cm^3	NA	-	-
R5	Theoretical maximum density (Gt)	g/cm^3	NA	-	-
R6	Draindown	%	< 0.3	Minimum point	[33]

2.7. SCANNING ELECTRON MICROSCOPY (SEM)

SEM (TM4000Plus with a Bruker SDD-EDS detector) was used to compare the physical microstructure features between CC and Viatop [61,62]. The fibre samples are mounted, then applied with a 15nm gold-coating by Emitech K550x Gold sputter coater.

3. RESULTS AND DISCUSSION

3.1. Size Distribution

Figure 1:1 shows the images of shredded CC and Viatop pellets used in this study. Figure 1:1 a and b are images with a 30-time magnification of both materials, Figure 1:1-c and Figure 1:1-d are CC and Viatop SEM images respectively. The images show comparable physical properties, such as the fibre length and the width and fibre agglomerations (marked in the images as 2 and 3). The Viatop fibres appear to have greater dispersion compared to the CC. Figure 1:1-c also shows the thin layer of LDPE in the CC sample, marked as 1.

3.1.1. Bitumen Properties

The results from the viscosity and penetration tests are shown in Table 4. The penetration value of bitumen C320 before and after RTFO conditioning is 51.6 and 30.5 dmm, respectively. After adding 0.5 and 1% LDPE to bitumen C320, the penetration decreased to 45.8 and 43.6 dmm, and after RTFO conditioning, the values dropped further to 27.2 and 26.1 dmm, respectively. The LDPE modified samples are compared with the viscosity classes C320, C450, and C600 shown in
Table 4.

Table 4: Summary of the properties of the binder's properties

Properties	Results of this study			Australian bitumen-grading specification		
	Virgin C320	0.5% LDPE	1% LDPE	C320	C450	C600
Viscosity at 60° C (Pa.s)	367.6	414.4	438.6	260 – 380	Report	500 – 700
Viscosity at 135° C (Pa.s)	0.606	0.66	0.726	0.4 – 0.65	0.70 max	0.6 – 0.85
Penetration at 25° C (dmm)	51.6	45.8	43.6	40 min	Report	20 min
Viscosity at 60° C after RTFO (Pa.s)	772	898.7	963.7	Report	750 – 1150	-
Penetration at 25° C after RTFO (dmm)	30.5	27.2	26.1	Report	26 min	-

The results suggests that C320 modified with 0.5% LDPE is comparable to C450, whilst 1.0% LDPE lies between C450 and C600 classes.

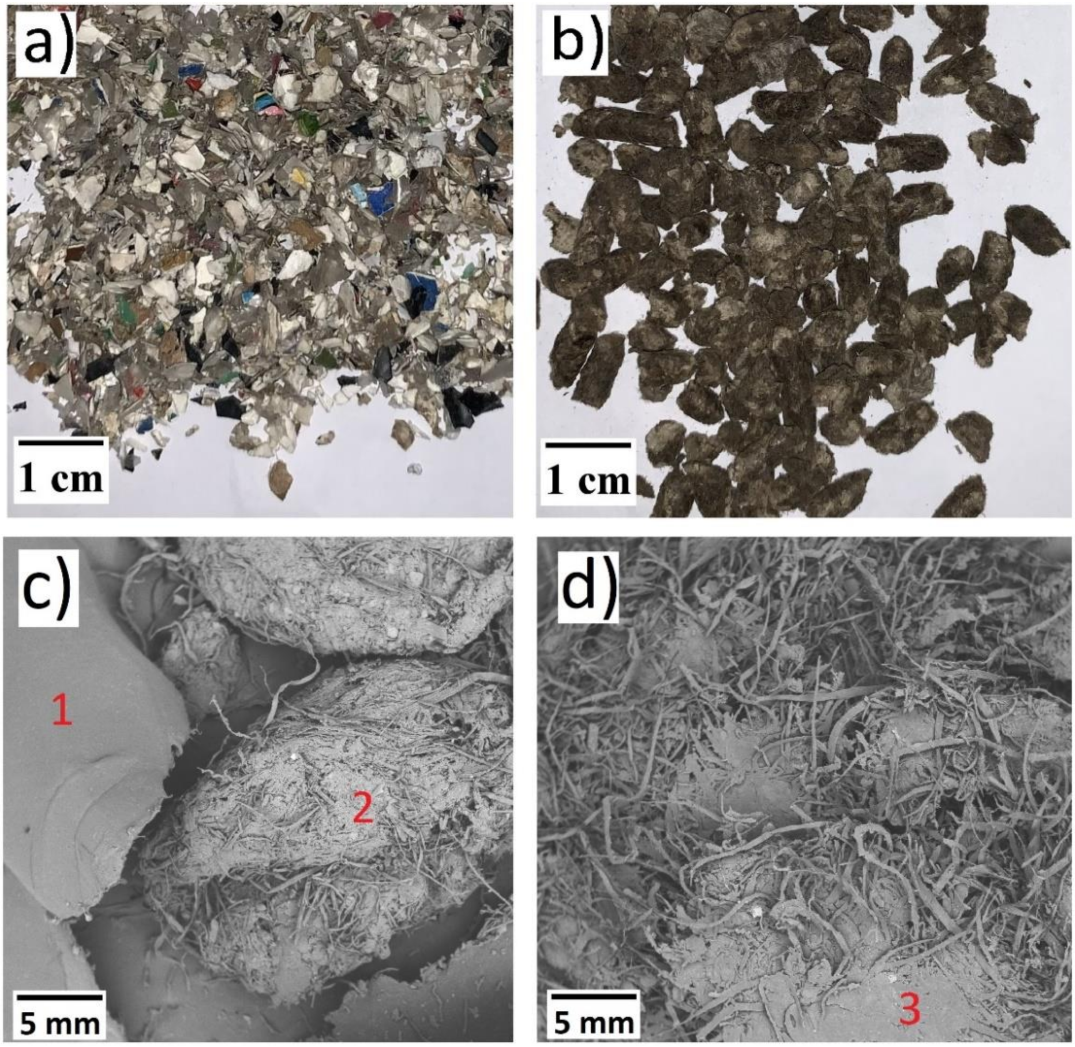

Figure 1: Photographs of a) CC fibres and b) Viatop fibres. SEM images of the c) CC fibres and d) Viatop. Number 1 is an LDPE particle in the shredded CC, 2 is a paperboard particle in shredded CC, and 3 shows the condensed fibres in Viatop

3.2. MARSHALL AND DRAINDOWN TEST RESULTS

Figure 2 shows the effect of CC and binder content on the Marshall properties of the mixture, including stability, flow, air voids (AV), and draindown. Increasing shredded CC % by mass increases stability and reduced flow number values compared to the control mixture. The former indicates increased stiffness, while the latter signifies decreased permanent deformation [41,63]. The Einstein relation describes that when particles are combined with a liquid the mixture viscosity increases [64] This impact may also be due resultant binder viscosity increase from LDPE modification and subsequent increased mixture stiffness [41].

Figure 2c illustrates that increasing CC % by mass increases mixture air voids (AV) for investigated binder contents. This is possibly to the fibre absorption of free binder, leaving larger inter-particle voids. The effect of LDPE on the viscosity also has impacts on the AV. Workability is inversely related to viscosity; the mixture is expected to compact less for equivalent compactive effort, leaving higher AV.

The draindown test results, depicted in Figure 2d, demonstrate that higher CC content corresponds to lower draindown. The most significant draindown reduction occurs between 0.3-0.4% CC are added. Impacts on draindown beyond 0.4% are limited; CC optimised % by mass is considered to be within 0.3-0.4%. Higher contents do not improve draindown performance and may introducing excessive stiffness.

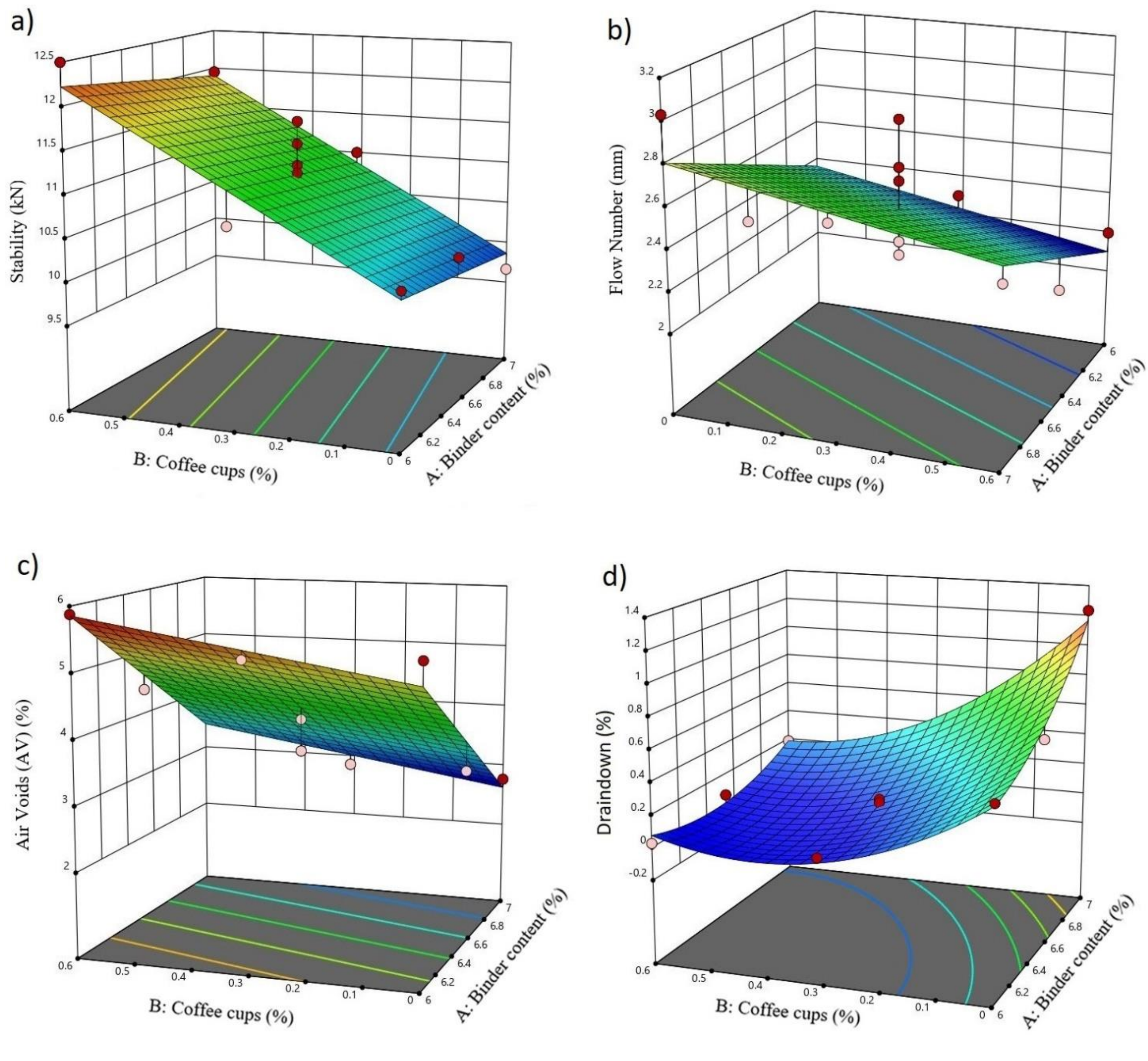

Figure 2: The effects of shredded CC and binder content on; a) stability, b) flow number, c) air voids, and d) draindown.

Table 5 tabulates the developed equations for stability, flow number, AV, and draindown. The coefficients suggest CC content has a relatively higher impact on stability and a lower impact on AV compared with binder content, whilst flow number is more influenced by binder content than the CC content. Draindown is more influenced by CC content than the binder content. Analysis of variance (ANOVA) results (P-values < 0.05) indicate factor significance.

Table 5: Predicted equations and Analysis of variance (ANOVA) for the experimental results (x_1= coded binder

content, x_2= coded CC content).

Responses	Coded equation	P-Values for the factors					
		x_1	x_2	x_1x_2	x_1^2	x_2^2	Model
Stability	$= 11 - 0.158 * x_1 + 1.02 * x_2$	0.323	< 0.0001	-	-	-	0.00012
Flow number	$= 2.46 + 0.22 * x_1 - 0.133 * x_2$	0.0325	0.17	-	-	-	0.0476
Air Voids	$= 4.24 - 1.22\ x_1 + 0.4 * x_2$	< 0.0001	0.0193	-	-	-	< 0.0001
Draindown	$= 0.101 + 0.206 * x_1 - 0.338 * x_2 - 0.149 * x_1\ x_2 + 0.123 * x_1^2 + 0.251 * x_2^2$	< 0.0001	< 0.0001	0.00063	0.0056	< 0.0001	< 0.0001

4. COFFEE CUPS COMPARED WITH VIATOP

Using the RSM method, the asphalt properties are optimised using the mix design parameters; 6.5%, 0.4% by mas CC. The results show [stability, FN, AV, Gb, Gt, and draindown] = [11.39kN, 2.41, 4.38%, 2.34, 2.45, 0.016%]. To evaluate the performance of the CC, the proposed solution is compared with Viatop. The design parameters of these two mixes are identical, with the exception of 0.4% CC and 0.3% Viatop by mass respectively. Flow, Gb, and Gt of both mixtures have high similarities (<0.1 mm difference FN and 0.02% for Gb and Gt). CC has higher AV and stability than Viatop. It is hypothesised that the increased stiffness and AV is due to LDPE modified binder CC material.

5. FIELD TRIAL - ON-GOING EVALUATION

Three field trials were conducted in Western Sydney between December 2022 and May 2023 under the NSW EPA Resource Recovery Framework to evaluate the field efficacy of CC fibres compared with Viatop in SMA10. The optimized fibre contents (0.3% Viatop and and 0.4 and binder percentages from this study were used over Trial 1 (100m private heavy vehicle roadway in Preston, NSW), Trial 2 (470m collector road in Penrith, NSW) and Trial 3 (330m residential road in Erskine Park, NSW). The trials used a control (SMA10 with Viatop) and treatment pavement (SMA10 with Coffee Cup Fibres) on identical Dense Graded asphalt base courses in continuous traffic lanes, as shown in figure 3.

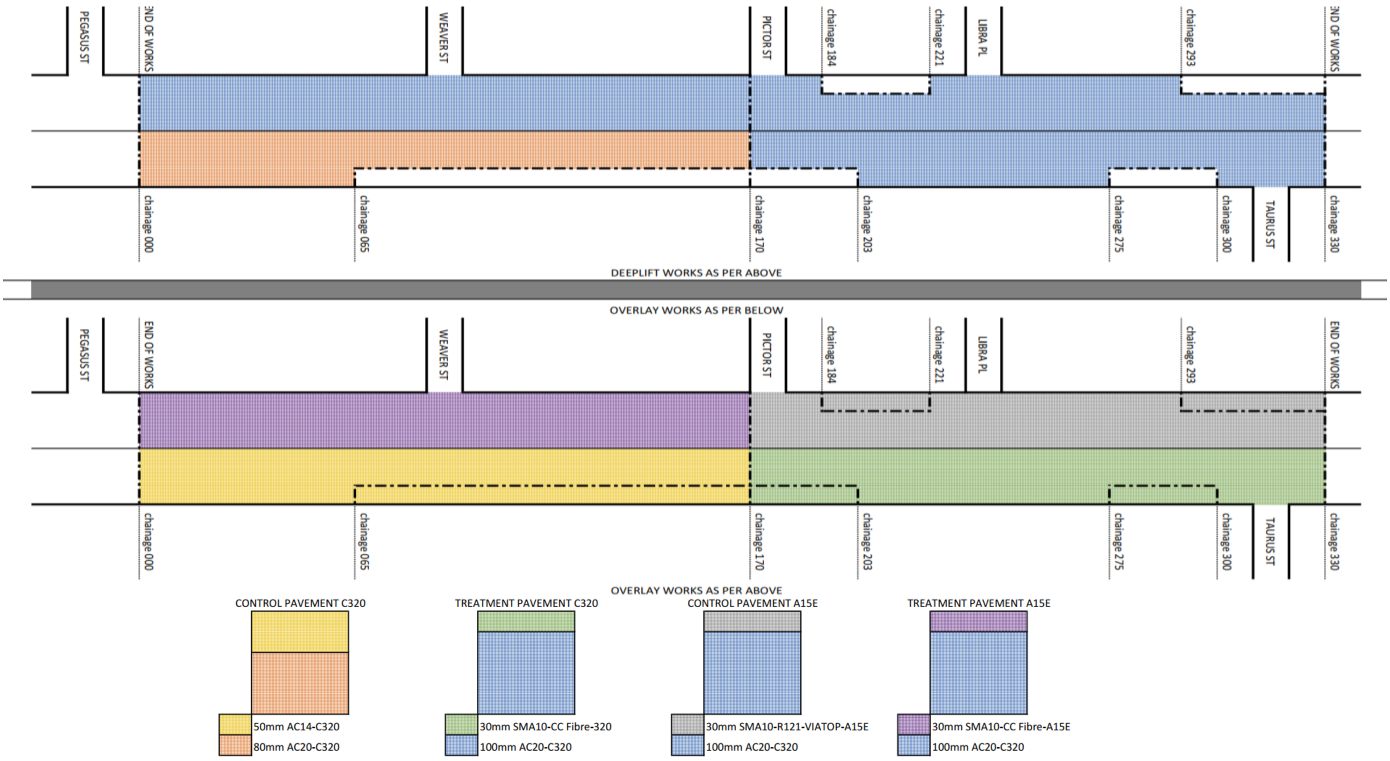

Figure 3: Pavement construction plan used to compare typical dense grade (14mm nominal) asphalt design with thinner SMA10 wearing course (Viatop control compared with CC fibre)

Table 6: Field trial volumetric properties and draindown results

Pavement Trial	Fibre type	Fibre Content	Binder Content (AS2891.3.1)	Air Voids (AS2891.8)	Draindown (TfNSW T648)
Prestons - 19/12/2023	CC	0.4%	6.3%	4.5%	0.10%
Prestons - 19/12/2023	Viatop	0.3%	6.5%	4.8%	0.00%
Penrith – 26/02/2023	CC	0.4%	6.5%	5.2%	0.15%
Penrith – 26/02/2023	Viatop	0.3%	6.5%	5.4%	0.10%
Erskine Park – 26/04/2023	CC	0.4%	6.3%	4.7%	0.10%
Erskine Park – 26/04/2023	Viatop	0.3%	6.6%	5.3%	0.05%

The field trials indicate comparable independent and dependent results to the laboratory testing. The draindown performance is consistent with the laboratory results; the Viatop mixes achieved lower draindown, however, Air Voids were not found to be predicted by binder content for constant Fibre content. This is likely due to minor variation within the asphalt particle size distribution across approximately 500T of SMA10 production across the trials.

6. CONCLUSIONS AND FUTURE WORK

This paper investigates the application of shredded CC as an asphalt additive. Marshall tests, draindown, SEM, and modified bitumen characterisation tests have been conducted, and the optimum shredded CC content is estimated. Based on the test results, the following conclusions are drawn:

- By introducing shredded CC into the mixture, stability and air voids increased, but the flow number dropped. Shredded CC also stiffened the mixture and enhanced resistance to permanent deformation. Based on the RSM method for SMA10 with 6.5% binder content, shredded CC content of 0.4% by total weight of the mixture met all specified limits in Australian Standards for stability, flow number, and air voids.
- Shredded CC significantly reduced the draindown of the mixture. The optimum amount of shredded CC content to prevent draindown was found to be 0.4% by the total weight of the mixture. Addition rates greater than 0.4% had negligible effect on binder draindown.
- Bitumen tests were conducted independently to simulate the impact of mild LDPE modification on the properties of the binder. Samples modified with 0.5 and 1% LDPE were examined for penetration and viscosity before and after RTFO aging and compared with virgin C320 binder.
- Properties of bitumen modified with LDPE with 0.5% LDPE plastic are comparable to C450 grade binder, whilst bitumen modified with 1% LDPE lies between the properties of C450 and bitumen C600 grade binder.
- SEM images show that the physical characteristics of CC and Viatop fibres are very similar; however, Viatop has greater dispersion than CC, and does not contain LDPE.
- The addition of 0.4% shredded coffee cups to the mix resulted in very similar performance to the commercial fibre (0.3% Viatop). The mixture with 0.4% shredded CC achieved higher stability and air voids which is likely due to the LDPE modification from CC.
- Given natural variation in particle size distribution in commercial asphalt production, further research using regression and optimisation approaches may provide deeper insight into predicting asphalt mix design properties for given input parameters.
- The treatment pavements will be monitored fortnightly for typical failure modes; cracking (over-stress), stripping (delamination and adhesion/cohesion impacts), and rutting (permanent deformation). The treatment pavements and control pavements will be assessed for wheel path rut depth using AustRoads Test Method AG:AM/T009 at annual intervals using a vehicle-based laser profilometer to assess permanent deformation over time.
- The field trial provides a promising demonstration of a viable recycling solution and end-market for hard-to-recycle liquid paperboard waste streams in construction materials.

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SUSTAINABLE CHOICES IN GEOTECHNICS: A CASE STUDY OF QUARRY TO PARKLAND CONVERSION

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ABSTRACT

Sustainability and sustainable development are broad concepts, and there is a growing imperative to both define sustainability, as per the 17 goals of the United Nations Division of Sustainable Development Goals (DSDG), and to regulate compliance with sustainable practice, such as the European Union's Corporate Sustainability Reporting Directive. The geotechnics practice, which is literally at the ground level of design and construction, has many opportunities to consider, develop and drive sustainability within our industry. This paper presents a case study of a quarry to parkland conversion project in suburban Sydney where sustainable practice was considered at every stage, from material reuse of existing fill to alternative means to reducing rock fall risk without installing support structures. The case study demonstrates how elements of sustainable practice in geotechnical engineering and engineering geology were achieved through comparison with select goals as published by the DSDG. Comparisons and contrasts are also made with other projects where perhaps a sustainable outcome could not be achieved due to factors such as existing Standards or time constraints. The paper summarises some of the difficulty of taking sustainable theory into practice and highlights how sustainable construction is often linked to the most economically viable design and maintenance solution. It is hoped that this paper will add to the growing industry knowledge of sustainable geotechnics in practice and provoke discussion of how to incorporate sustainability within the context of our current framework of Standards and standard industry good practice for design.

1 INTRODUCTION

Sustainability and sustainable development are broad concepts – sustainability itself represents the ability of a system to continue to function over time without loss of effectiveness; where supply meets or exceeds demand over a given timeframe (Basu, et al., 2015). Sustainable development is considered to have been best defined by the Brundtland report produced in 1987, which noted that sustainable development was "development that meets the needs of the present without compromising the ability of future generations to meet their own needs" (Basu, et al., 2015; Lagesse, et al., 2022). This lead to the formation of a United Nations programme for sustainable development, now represented by the Division for Sustainable Development Goals (DSDG) (Lagesse, et al., 2022; United Nations, 2023). In 2015, the DSDG formulated the 2030 Agenda for Sustainable Development, with 17 Sustainable Development Goals that attempt to address the core ideals behind sustainable development – i.e., allowing for development that does not compromise future generations (United Nations, 2023).

All member nations of the UN have signed up to these 17 goals (and their 169 targets split across the goals) (United Nations, 2023), and thus many government authorities have introduced or developed regulations to measure compliance with sustainability targets. An example of this is the European Union's Corporate Sustainability Reporting Directive which requires companies of a certain size to disclose information on risks and opportunities related to social and environmental issues within their areas of working (European Comission, 2023). Another example is the increasingly common field of Environmental, Social and Governance (ESG) consulting, as corporations and regulators look to improve their internal and external alignment with sustainable activities (UN Environment Programme - Finance Initiative, 2004).

The geotechnics practice is literally and figuratively at the ground level of design and construction and is often most heavily involved at the early stages of a project. This position gives geotechnics a high level of impact on the overall sustainability of a project (Basu, et al., 2015; Clayton & Smith, 2015; Gill, 2017; Lagesse, et al., 2022; Pantelidou, et al., 2012; Purdy, et al., 2022). Prior to the release of the 17 SDGs in 2015, multiple studies were undertaken to identify how sustainable development could be embedded in civil engineering, both in construction and consultancy, some with a focus on environmental sustainability and the reduction of greenhouse gas emissions (Pantelidou, et al., 2012; Costigane & Guthrie, 2013; MacAskill & Guthrie, 2013). Other publications looked at the role of geotechnical engineering within the context of sustainable development (with social and economic sustainability also forming part of the discussion) (Basu, et al., 2015) including site investigation impacts (Clayton & Smith, 2015).

In 2017, the role of geology in achieving the 17 SDGs was explored, presented as a matrix to identify the broad contributions to each individual goal by relevant disciplines including hydrogeology and engineering geology (Gill, 2017). This was later refined specifically for engineering geology in a detailed mapping exercise by Lagesse, et al. (2022) to demonstrate how engineering geologists could support all 17 SDGs, and the majority of the individual targets within those goals, though some were indirect contributions. The role of engineering geological practice in achieving goals such as SDG 1 No Poverty and SDG 2 Zero Hunger (as examples) was demonstrable, but indirect. By comparison, arguably the strongest contributions of engineering geological practice were to SDG 7 Affordable and Clean Energy, SDG 9 Industry, Innovation and Infrastructure, and SDG 11 Sustainable Cities and Communities (Lagesse, et al., 2022).

Based on the above, one of the largest impacts that geotechnics has on sustainable development is through construction, from project inception to design to construction to operations. Sustainability is often driven by geotechnics through effective site investigations which provide ground information to de-risk design and construction (Clayton & Smith, 2015) and by considered design that reduces movement of material, makes use of existing ground conditions (Chen & McIlquham, 2023), and minimises built infrastructure and construction impact on communities (Basu, et al., 2015). These activities also contribute to improving the resilience of a project, which represents the ability of the system to withstand or adapt to outside stresses or environmental changes (most often considered for climate change). Therefore, it can be seen that many opportunities existing within the geotechnics profession to drive sustainability within many industries. However, there are many challenges which must be considered and overcome to balance reliable and robust designs that meet risk mitigation requirements with sustainable practices.

2 THE QUARRY

The site is a former road-base aggregate quarry situated within a volcanic breccia/basalt intrusion, hosted within sandstone. The quarry was operated intermittently for several decades, and developed such that the base was over 120 metres below the original surface. After quarrying ceased, the site was acquired by the local government, which then allowed it to be used as a storage for spoil from a nearby, significant tunnelling project. This filled the void up with nearly 55 vertical metres of crushed sandstone and shale, which was placed via conveyor belt and pushed into place by front end loaders.

In order to make use of the land, the local government council developed a proposal to convert the site into a parkland for the benefit of the local community, to remediate the natural environment, preserve the cultural, geological and industrial heritage of the area, and to create a drawcard for tourism. This parkland conversion presented a challenging geotechnical and engineering geological puzzle; what could be done to treat the rock fall risk associated with the still exposed quarry walls, the long-term settlement of the spoil within the quarry void, and the slope stability of the adjacent waste dumps and infrastructure pads, which had since become heavily vegetated. This case study will focus on the work undertaken for addressing geotechnical risks associated with the highest wall within the quarry, and how the choices aligned with a sustainability perspective, and also presents (at a higher level) those choices made for the deep quarry spoil and the other quarry high walls and surrounding slopes.

The aforementioned highest wall within the quarry is approximately 70 metres high, and is comprised of a relatively fresh, medium to high strength rock for the lower two-thirds, with a highly weathered, near soil-strength weathered rock within the upper third. The lower, fresh rock zone is comprised of three faces (with two benches in between) with a face angle of approximately 80° from horizontal. The upper face has no benches, and has either been laid back to approximately 50° or has "fretted" back to this profile over time. The high wall appears to have weathered noticeably and been subject to slopewash since exposure, as inferred from built up colluvium on the lower benches, and small rocks at the toe that are surmised to have fallen from above.

3 GEOTECHNICS DESIGN CHOICES

A key component of the parkland conversion was to allow public access to the quarry void, and thus the presence of the public at the toe of each high wall and slope had to be considered. A typical methodology for treating geotechnical risk of this kind would be to cut back the upper, weathered material to a shallower profile, and potentially consider a shotcrete facing and/or soil nails. For the lower, fresher rock mass, the methodology might be to introduce draped or anchored mesh, secured with rock bolts in a systematic pattern, and individual blocks further secured with additional rock bolts. Such works would entail a significant financial cost, but also require consumption of manufactured materials like cement and steel. These two elements alone carry a large environmental burden; a tonne of steel produces 1.83 tonnes of CO_2 emissions (Pandit, et al., 2020), while a tonne of cement produces at least a tonne of CO_2 (United Nations Environment Programme, 2010). Such systems introduce a potentially onerous maintenance regime, depending on constructability, access and scale, and there is often minimal resilience in these systems.

For the highest wall in the quarry, it was not considered economically viable nor aesthetically pleasing to install slope stabilisation treatment. However, the risk to the public had to be considered. Therefore, rock fall trials were conducted, whereby "adverse case" boulders were dropped from the crest of the high wall (via excavator) and the distance to which the boulders rolled from the toe recorded. The findings of these trials supported the concept of an exclusion zone at the toe of the wall, that would negate the requirement for built infrastructure. Council had planned a wetland area within the parkland to serve as a natural water filtration system for the man-made lake planned in the quarry. The opportunity to combine the exclusion zone with the wetland system was thereby proposed, as the wetland would be excluded to public access while also serving to capture incidental rock fall.

Some of the benefits of using a wetland as an exclusion zone are:

- The ground is muddy, thickly vegetated and has numerous small waterbodies, which make traversing it less appealing to the general public.
- Maintenance of such a feature is much lower than many other types of exclusion zone (for example, any incidental rock fall does not need to be cleared as frequently).
- The coefficient of restitution is lower in such an environment than for cleared ground, talus or gravel (Hoek, 2023). This significantly reduces runout distance (and bounce height) of falling material, which can lead to a thinner exclusion zone, if needed.

The stability of the upper, weathered portion of the high wall was improved through diverting surface water run-off and retaining the vegetation at the crest (largely comprised of well-established, endemic native plants), with a view to increase the amount of native ground cover established on the face, as well. This had a positive environmental impact as well as improving the resilience of the design (i.e., a design solution that is more tolerant of climate change and that does not introduce built infrastructure with a fixed design life). It is important to note that limited devegetation was undertaken on the lower, fresh rock zone so as to reduce the risk of root jacking (i.e., the vegetation was not considered to be stabilising the face).

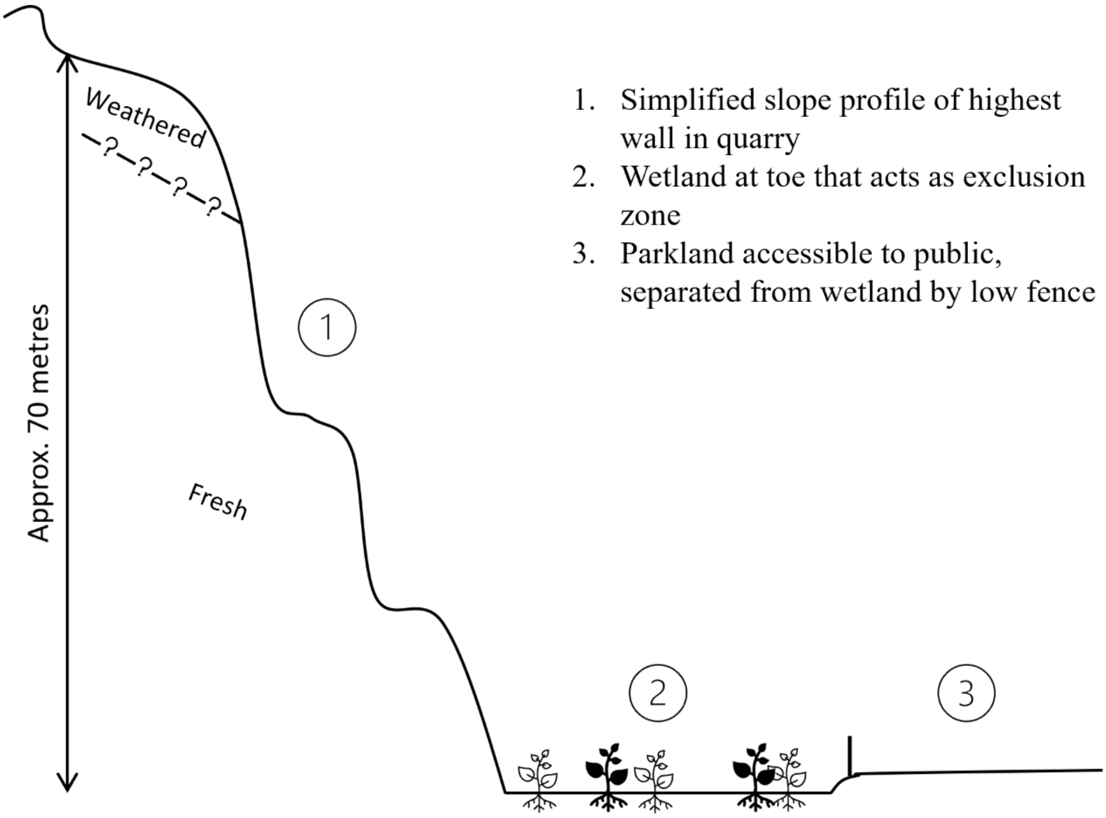

Figure 1: Indicative illustration of the risk mitigation measure of the high wall

Elsewhere in the quarry void, more typical slope stabilisation treatments were applied. This included selected rock bolts to retain individual blocks, and systematic draped mesh or anchored mesh. The decision to undertake slope treatment in these areas was based on a combination of the assessed risk requiring some form of rock fall hazard mitigation, and a lack of space at the toe for an exclusion zone. Modifications to the design, over time, introduced potential exclusion zones (such as where a road embankment created an elevation difference between the public and the toe of the wall, to allow for a “catch ditch”), and these were generally adopted to negate the requirement for mesh installation. Indeed, catch ditches are noted as being an effective method of reducing rock fall risk (Hoek, 2023).

The slopes around the quarry are generally waste material from quarry operations or overburden, dumped out on top of the residual soil and colluvium of the original valley. These slopes have become overgrown with native and non-native trees in the time since quarry operations ceased, but do not have established ground cover, indicating the lack of soil within the waste material. Quantitative and Qualitative Risk Assessments (collectively known as QRAs)were undertaken in accordance with the guidelines presented by the Australian Geomechanics Society Landslide Taskforce, often referred to as AGS 2007c (Walker, et al., 2007), using anecdotal evidence from Council employees and observations of slope failure such as talus and scour to determine likelihood. These QRAs determined that the overall slope risk was within the acceptable risk level suggested in AGS 2007c for existing slopes (i.e., less than 1×10^{-4}), though the potential for large failures to occur during adverse rain fall was identified.

Noting this, it was identified that slope risk would increase further if vegetation were removed, as the trees were beneficial to slope stabilisation by binding together the loose boulders that formed the "skeleton" of the dumped spoil. Administrative and isolation controls were put in place to mitigate the risk of large slope failures based on rain fall (i.e., trigger levels for rain fall in a certain period that would require complete closure of the parkland, and follow-up inspections prior to re-opening). These controls negated the requirement for potentially extensive clearance and retaining structures adjacent to access roads and walking tracks within the site.

The quarry void "backfill", as previously mentioned, was some 50 metres of crushed tunnel spoil, which had been loosely placed via conveyor belt and front-end loader. No specified compaction method had been undertaken to improve the backfill as it was placed. In addition to this, the material had been subject to several cycles of inundation by surface and groundwater (the quarry being below the water table). The parkland conversion involved creating a new landform that would reshape this backfill and incorporate cut material from elsewhere in the scheme. The final landform would also feature several single-story buildings and other lightly loaded structures and pavement, though exact placement was not yet determined. Therefore, a traditional "structural fill" capable of providing adequate bearing capacity with minimal risk of differential settlement was required.

Tunnel spoil in the Sydney basin is noted as being a suitable fill material for construction (Chen & McIlquham, 2023) and was therefore planned to be structural fill. A specification for structural fill based on the engineering parameters of this material was created, to inform re-use of site won material elsewhere as well. However, the desire was to have less suitable material (such as oversize boulders, broken concrete slabs, and clayey material) won on site placed within the void as well, to avoid transporting these less suitable materials off site. In order to achieve this cut-fill balance, a pragmatic approach was undertaken whereby these less suitable materials were placed as basal fill, to a maximum compacted depth and with a minimum thickness of structural fill above. Boulders (up to 0.6 m) were to be placed such that they were kept from forming vertical stacks, but otherwise allowed to be used in the basal fill. This reduced the material handling, processing and transport of the landscaping aspect of the parkland conversion. The basal fill / structural fill arrangement was subject to successful field trials to demonstrate its viability as a design solution.

A limited site investigation was undertaken to inform design elements as well as assess the physical characteristics of the tunnel spoil within the quarry void. The term limited is used here specifically as the decision was made to gather as much information as possible while having a cost-effective geotechnical investigation. To achieve this, specifically targeted intrusive investigations (boreholes, test pits and Cone Penetrometer Tests) were undertaken to inform the most important design elements, in consultation with Council. These important design elements were generally those that presented the largest risk to construction (based on delays or material costs) unless geotechnical uncertainty was reduced. Other design elements were inferred from nearby or relevant geotechnical and geological observations and testing.

4 DISCUSSION

2.1 DESIGN ELEMENTS IN CONTEXT OF SUSTAINABILITY

The below table (Table 1) demonstrates how the design elements of this case study map to the SDG targets, with additional commentary on where the measures sit on the Hierarchy of Controls, similar to the work presented in Lagesse, et. al. (2022). It can be seen that **SDG 11 Sustainable cities and communities** is not specifically demonstrated in Table 1. Rather, the work performed generally aligns with 11.4 Protect and safeguard cultural and natural heritage, demonstrated by the role that geotechnics played in facilitating the parkland conversion in a constructible and economically viable manner.

Table 1: Mapping of design elements to SDG targets and their Hierarchy of Control types

Design Element	Relevant SDG Targets	Evidence of sustainable choices	Hierarchy of Control Type(s)
High wall stabilisation / geotechnical risk mitigation	**9 Industry, innovation, and infrastructure** 9.1 Sustainable and resilient infrastructure. 9.4 Upgrade and retrofit infrastructure and industry to enhance sustainability. **12 Responsible consumption and production** 12.5 Reduce waste generation through prevention, reduction, recycling and reuse.	Establish exclusion zone at toe of the high wall to negate installing slope stabilization. Resilience is achieved through the system remaining valid even if rock fall occurs.	Isolation, Engineering Controls
Basal fill	**9 Industry, innovation, and infrastructure** 9.1 Sustainable and resilient infrastructure. **12 Responsible consumption and production** 12.5 Reduce waste generation through prevention, reduction, recycling and reuse.	Reduction in vehicle movements and haulage of fill material, thereby reducing fuel consumption, potential dust escaping to atmosphere, and noise. Reuse of existing fill material and potentially less desirable, site-won fill to achieve cut-fill balance.	Engineering Controls
Surrounding slope stabilization	**9 Industry, innovation, and infrastructure** 9.1 Sustainable and resilient infrastructure. **12 Responsible consumption and production** 12.5 Reduce waste generation through prevention, reduction, recycling and reuse.	Utilise stabilising effect of existing vegetation and introduce park closure regimes to isolate the public from potential hazards based on rainfall trigger levels.	Isolation, Administrative Controls
Limited site investigation	**12 Responsible consumption and production** 12.4 Environmentally sound management of chemical and all wastes throughout their life cycle. 12.5 Reduce waste generation through prevention, reduction, recycling and reuse.	Specifically targeted boreholes and test pits, with intent to inform most critical design elements as well as tangentially inform other elements, thus reducing fuel consumption. Remove all drilling consumables and wastewater from site.	Engineering Controls (though this does not directly translate to the Hierarchy of Control as it relates to *design risk* rather than *slope risk*).

The design solutions presented above were often the most cost-effective means for addressing the risk associated with the individual items. They were also in general accordance with current industry practice for treating geotechnical or civil engineering risks, though with some departures to reduce both cost and environmental impacts. This was a fortunate case in which the geotechnical design solutions were demonstrably aligning with both a sustainable outcome and current industry practice for risk assessment and risk mitigation. However, there are some difficulties aligning sustainability, resilience and industry standards for risk management / geotechnical design (Basu, et al., 2015), as is explored in the following sections.

2.2 HIERARCHY OF CONTROLS AND RESILIENCE

As noted previously, resilience in design refers to the ability for the design intent to be remain valid during stresses or in the event of failure, as measured through robustness and redundancy (being tolerant of breakage or damage), resourcefulness (identification of or adaptability to disruption) and rapidity (the speed of recovery of the design to stresses or failures) (Das, et al., 2018).

When viewed from the perspective of applying the hierarchy of controls then, in the author's opinion, resilience may therefore best be achieved via elimination, isolation or administrative controls for slope stabilisation / risk mitigation. For example: elimination may be achieved through regarding the slope to a more stable profile; isolation may be achieved through development of exclusion zones, and administrative controls can take the form of trigger levels, procedures, and signage. These mitigation measures can be designed to "bounce back" and remain just as effective in the event of failure (MacAskill & Guthrie, 2013).

However, elimination (i.e., removal of the hazard) can often be more expensive and environmentally impactful than isolation or administrative controls. The benefit of elimination is that it offers the greatest possible risk mitigation, hence its position at the top of the Hierarchy of Controls, which ranks risk mitigation types from most to least effective (Worksafe Victoria, 2023). However, a sustainable outcome might be best achieved by isolation or administrative controls, when consideration is given to the mitigated risk level, and whether this reduced risk (without elimination) is broadly acceptable to the asset owner, in line with their own risk management regime and any relevant standards.

2.3 CONSIDERING RESILIENCE AND SUSTAINABILITY WITH RISK MANAGEMENT

The overall goal of any risk management is to reduce the risk to nil, or at least to a 'tolerable level' depending on the risk appetite of the asset owner or vulnerable parties (Walker, et al., 2007). The AGS 2007c Practice Notes for Landslide Risk Assessment (Walker, et al., 2007) define As Low As Reasonably Practicable (ALARP) as being a pragmatic approach for achieving a tolerable level of risk mitigation, with further reduction no longer being practical in cost to an individual or asset owner. The "test" for what is reasonably practicable is therefore linked to economic viability of further risk reduction, but does not have a direct link to "sustainability", that is to say, at which point is the proposed overall solution or design at odds with the sustainability targets of the SDG or even those of the asset owner.

As a high level example, a theoretical scenario is as follows: a popular walking track sits below a steep slope with a risk of rock fall and translational slide. A quantitative risk assessment finds the most vulnerable person (hikers stopping to take photographs for 30 minutes) as having a risk to loss of life (LoL) of 1×10^{-3} without control measures. A qualitative risk assessment on damage to property finds that the risk of the walking track infrastructure being lost due to translational slide as $1 \text{ x } 10^{-4}$ (i.e., 1 in 10,000) without control measures.

- Individual block removal via hand tools brings LoL risk down slightly to 5×10^{-4}.
- Rock bolting blocks that cannot be removed brings LoL risk down to 1×10^{-4}.
- Widescale scaling of crest to reduce slope angle, and remove non-native trees causing root jacking, brings the slope risk level down to 5×10^{-5}.
- It is within the budget of the asset owner to undertake these activities, as the location is near the start of the walking track, and the track's carpark provides a suitable staging ground.
- However, these last two measures add to the carbon footprint of the risk reduction, via factors such as grouting, fuel usage, additional spoil removal, etc. Tree removal also impacts both environmental and community sustainability. In addition, community is impacted further by closing the walking track for a longer construction programme.
- In this instance, ALARP could allow for this impact to overall sustainability of the walking track, but if the sustainability is considered, it may be more "practicable" to undertake individual block removal, and place signage indicating the risk of rock fall and slope failure, and not to pause to take photographs. Allowing trees to grow up to block the view may help. In addition, the asset owner may wish to close the hiking track during inclement weather which may trigger the failure.
- Note that, if the slope were further down the walking track, it may be economically unfeasible to undertake the remediation; an example of how economics can be linked to sustainable choices.

Although hypothetical, the above example happens across the industry on a regular basis. Risk assessments are generally quantitative or at least provide a numerical outcome (based on a risk matrix) once probability and consequence have been assigned (in the case of qualitative analysis). Therefore, the inclusion of sustainability and resilience factors may not seem readily achievable. Previous work by others (Das, et al., 2018; MacAskill & Guthrie, 2013) has identified the value in conducting Multi-Criteria Analysis (MCA) as a means to embed sustainable and resilient design outcomes in a typical risk management workflow. In the realm of slope stabilisation, with respect to the guidelines provided in AGS 2007c, a potential workflow might look like Figure 2 below.

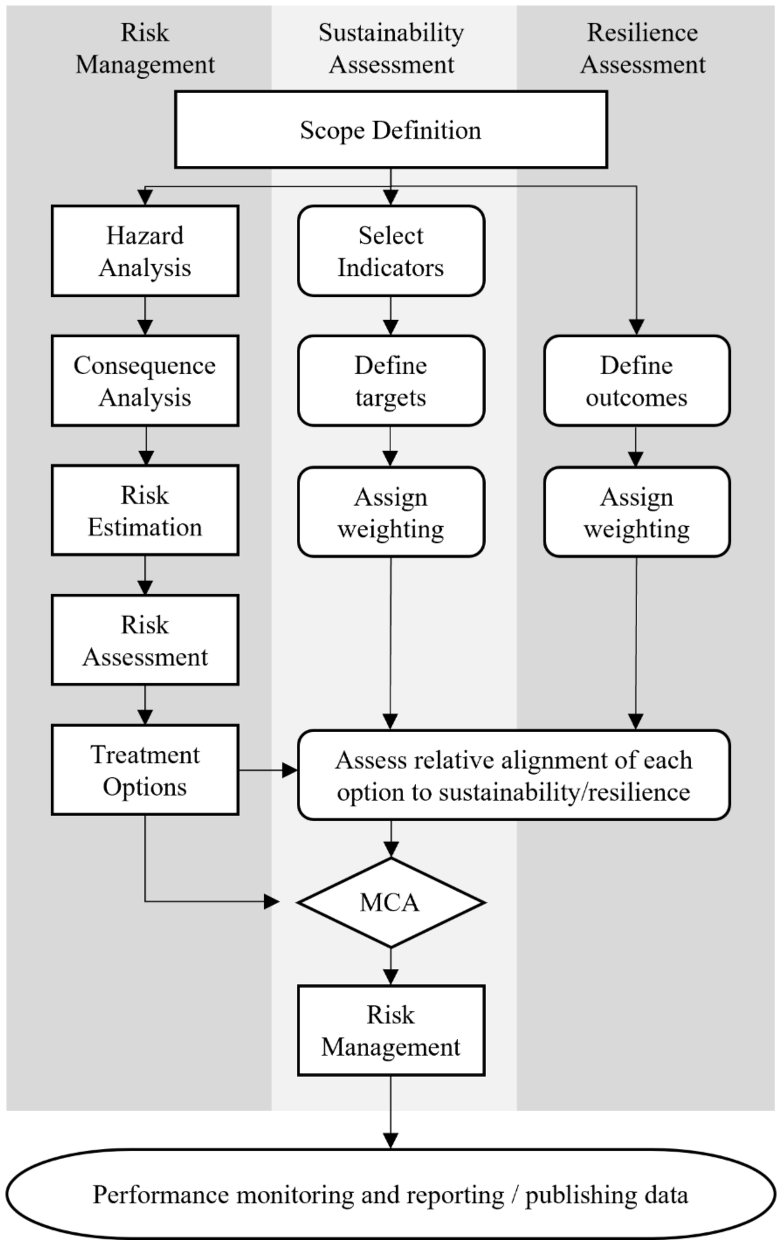

Figure 2: Potential workflow for considering sustainability and resilience in design along with appropriate risk estimation and mitigation, adapted from (Walker, et al., 2007) and (MacAskill & Guthrie, 2013)

Guidance on the selection of sustainability and resilience outcomes, and appropriate weighting, are beyond the scope of this paper, though other publications provide examples, such as (Das, et al., 2018) and (MacAskill & Guthrie, 2013). The monitoring of performance of the risk mitigation measures, resilience over time, and the conformance to sustainability targets, is a vital step in iteratively improving the embedment of sustainability in geotechnics. Although the above flowchart refers to improving the sustainability of a slope stabilisation solution, there is another area where the geotechnics practice can significantly contribute to a better outcome for sustainability and resilience in design in general (Clayton & Smith, 2015).

2.4 SITE INVESTIGATIONS AND IMPACTS ON PROJECT SUSTAINABILITY

Site investigations have arguably the greatest impact on sustainability of all the activities that are carried out by the geotechnics practice. In addition, many projects require regular intrusive investigation locations to meet the specifications of a given type of built infrastructure. The intent behind a comprehensive intrusive investigation is to de-risk the design, which theoretically will reduce construction time or material consumption (Clayton & Smith, 2015). Therefore, such regular investigations may also be more sustainable when reviewing the potential impacts of a project across its lifecycle (known as an LCA or Life Cycle Assessment), despite the initial impact on the sustainability profile (Purdy, et al., 2022).

This places some onus on the consulting engineering geologists and geotechnical engineers to design site investigations that provide as much information as possible from the fewest individual investigations, or to gather data through more sustainable methods (such as geophysical surveys which use less fuel and consumables like cement than traditional borehole drilling (Clayton & Smith, 2015; Purdy, et al., 2022). However, there is also a need for engagement with, and by, the asset owner or end client on where geotechnics can improve the project's sustainability (or resilience), either in design, construction or operation (Pantelidou, et al., 2012). As an example (in the author's experience) there is often a requirement to design "one size fits all" solution which accounts for the potential adverse case, independent of the level of knowledge of the ground that has been achieved. Early collaboration may lead to outcomes which de-risk the design while also positively impacting the sustainability of the project overall, such as the aforementioned case for improving the knowledge of the ground and thus reducing the construction materials. Initiatives such as the comprehensive and sustainable GIS database for Australia (Och, 2023), similar to those seen in other countries, will improve the knowledge of ground conditions without undertaking comprehensive site investigations. However, support from asset owners to allow reliance on these data (gathered by third parties) is a necessary step to reducing the sustainability impact of site investigations.

5 CONCLUSIONS

This paper summarises how select choices to address geotechnics challenges improved the sustainability and resilience in of the design outcomes, partly by mapping the solutions against the UN Sustainable Development Goals. It also highlights some of the difficulties with taking a "sustainable approach" while also applying current industry practice for risk mitigation. It notes the importance of early engagement between geotechnics professionals and asset owners or clients to improve overall sustainability of a project throughout its lifecycle. As government regulation of compliance to sustainable targets is progressively rolled out, the need to consider resilience and sustainable development in design (through MCA or options assessment) will become both commonplace and critical. Therefore, it is important for the geotechnics practice to develop workflows to balance the need for robust and reliable design with resilient and sustainable development.

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OPPORTUNITIES FOR SUSTAINABLE GEOTECHNICAL ENGINEERING PRACTICE: TWO CASE STUDIES FROM AUSTRALIA

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ABSTRACT

Geotechnical design elements are major consumers of energy and natural resources in civil and infrastructure projects. However, by applying appropriate levels of engineering and investigation, opportunities exist for more sustainable and resilient solutions that better understand the ground and geo-structure characteristics. This paper presents two project examples from Australia to illustrate such opportunities.

The first example is a residential complex development project proposed over a reclaimed land with underlying soft marine deposits in Sydney. An advanced site investigation revealed an opportunity to use preload and surcharge ground improvement methodology instead of the initially proposed extensive piled slabs. The earthworks were nearly completed, and geotechnical monitoring confirmed the design assumption. A project-specific earthwork specification was developed, considering the site won material characteristics and performance requirements of different zones of earthworks. Through collaboration between the designer and contractor, most of the site-won material could be used in specified zones of the earthworks on site, with or without treatment, without compromising the performance requirement.

The second example is a major new highway project, where value engineering led to a review of the concept design bridge foundations, retention systems, and site investigation results. Alternative structure types and amendments to the earthwork specification were recommended and justified by the local road authority. The proposed changes significantly reduced the construction cost and carbon footprint of the project, making it viable for ministerial approval. The paper presents a comparison of the CO emissions between the alternative solutions proposed and those initially considered, highlighting the contributions made in these projects to global and local sustainability goals.

1 INTRODUCTION

In an era marked by environmental challenges and the urgency of climate action, the importance of integrating sustainability principles into civil engineering projects cannot be overstated. Civil projects, often characterized by their substantial resource consumption and long-lasting impact, have a significant role to play in shaping a more resilient and environmentally conscious future. One of the key aspects of sustainability is the reduction of carbon footprint, which directly addresses the critical issue of climate change. By minimising carbon emissions throughout the lifecycle of a project, from construction to operation and eventual decommissioning, civil engineering can significantly contribute to global efforts to mitigate climate change. Embracing sustainable practices not only demonstrates a commitment to environmental responsibility but also yields economic benefits through enhanced efficiency, reduced operational costs, and increased project longevity. The incorporation of sustainability into civil projects is no longer a choice but a necessity to safeguard our planet for current and future generations.

In this paper, some of the global and local sustainability goals requirements are explained, some of the example methods to contribute to these goals in civil and, in particular, geotechnical engineering practice are materialised, and then two project examples from Australia are presented that exemplify how geotechnical engineering practices can align with the global and local sustainability goals, contributing to a more sustainable and resilient built environment. These examples demonstrate how considering ground and geo-structure characteristics can lead to innovative and environmentally conscious solutions addressing local and global challenges.

2 GLOBAL AND LOCAL SUSTAINABILITY GOALS

2.1 UNITED NATION SUSTAINABILITY GOALS

The United Nations (UN) has recognised the importance of sustainable development and has set forth 17 Sustainable Development Goals (SDGs) as part of the 2030 Agenda for Sustainable Development. These goals aim to address global challenges, including poverty, inequality, climate change, environmental degradation, peace, and justice (UN, 2023). Civil engineering projects have a significant role in contributing to several of these goals. Geotechnical engineering, being a vital aspect of infrastructure projects, can contribute to achieving these goals. The following SDGs out of 17 are listed below, which, to the authors, civil and geotechnical engineers could contribute most:

- Goal 6: Clean Water and Sanitation
  Civil engineering projects can promote access to clean water and sanitation, ensuring adequate water supply and wastewater treatment for communities. Efficient water management, rainwater harvesting, and sustainable drainage solutions, using recycled water sources for the construction of infrastructures are among the approaches that can support this goal.

- Goal 7: Affordable and Clean Energy
  By incorporating renewable energy sources and energy-efficient technologies, civil engineering projects can help promote affordable and clean energy access, reducing greenhouse gas emissions and mitigating climate change. Geotechnical engineers play a crucial role in achieving this goal by providing tailored geotechnical solutions for various infrastructures. This includes designing more sustainable foundations for onshore and offshore wind turbines, optimizing setups for solar farms, enhancing dam structures for hydroelectric power, and harnessing wave energy. Their expertise ensures that these energy projects are not only efficient but also environmentally responsible, contributing significantly to the global pursuit of affordable and clean energy.

- Goal 9: Industry, Innovation, and Infrastructure
  Infrastructure development is at the heart of Goal 9, and civil engineering projects can contribute by creating resilient, sustainable, and inclusive infrastructure that supports economic growth and societal needs, as further discussed in this paper.

- Goal 11: Sustainable Cities and Communities
  Civil engineering plays a crucial role in designing and developing sustainable urban areas, including efficient public transportation, green spaces, and resilient infrastructure, which are all essential components of sustainable cities and communities.

- Goal 13: Climate Action
  Addressing climate change is a top priority, and civil engineering projects can contribute by adopting climate-resilient design and construction practices, promoting low-carbon technologies, and implementing climate adaptation measures, as further exemplified in this paper.

- Goal 15: Life on Land
  Infrastructure development often involves land-use changes. Civil engineering projects can contribute to preserving biodiversity and ecosystems by adopting sustainable land-use practices and integrating green infrastructure.

2.2 INFRASTRUCTURE RATING SCHEME

Various regions around the world have established specific requirements and classifications to ensure sustainable practices in civil engineering projects. In Australia and New Zealand, the Infrastructure Sustainability Council (ISC) has played a leading role in setting standards and guidelines for sustainable infrastructure projects.

The ISC provides a comprehensive rating system known as the Infrastructure Sustainability (IS) Rating Scheme. This scheme assesses and certifies the sustainability performance of infrastructure projects across various sectors, including transportation, water, energy, and social infrastructure.

The IS Rating Scheme evaluates projects based on various sustainability categories, covering aspects such as governance, environmental impact, economic viability, and social outcomes (ISC,2023). The key categories typically include:

- Management and Governance: This category assesses the project's commitment to sustainability, including the implementation of sustainable policies, resource management, and stakeholder engagement.
- Environmental: The environmental category evaluates the project's impact on natural resources, biodiversity, greenhouse gas emissions, and water management, encouraging practices that minimize environmental harm.
- Economic: The economic category considers the project's life cycle cost analysis, economic efficiency, and value for money, ensuring that long-term financial benefits are considered.
- Social: Social aspects, such as community engagement, safety, and workforce welfare, are evaluated to promote projects that benefit and engage with local communities.

- Innovation: This category recognises projects that incorporate innovative solutions and technologies to enhance sustainability and efficiency.
- Liveability: Focusing on the end-users, this category assesses the project's impact on improving the quality of life for people who use or are affected by the infrastructure.

The IS Rating Scheme provides a rigorous framework for evaluating the sustainability credentials of infrastructure projects in Australia. It encourages stakeholders to adopt sustainable practices, promote innovation, and deliver long-term benefits for the community and the environment.

For civil engineering projects in New South Wales (NSW), compliance with ISC's sustainability requirements is becoming increasingly important for gaining approvals and achieving positive public perception. By adhering to the IS Rating Scheme, projects can demonstrate their commitment to sustainability and align with global goals, such as the UN's Sustainable Development Goals.

3 HOW CAN CIVIL ENGINEERS/GEOTECHNICAL PRACTITIONERS DRIVE SUSTAINABILITY ACROSS A PROJECT LIFECYCLE?

In the realm of civil and geotechnical engineering practices, the pursuit of sustainability has evolved into a cornerstone of responsible design and construction. This commitment to sustainable practices unfolds across various project phases, from conception to completion, and draws insights from a wealth of literature and research in the field.

3.1 PLANNING FOR SUSTAINABILITY

In the pursuit of sustainability, geotechnical practitioners hold a pivotal role right from the initial planning phase. This critical stage acts as a guiding compass, laying the foundation for subsequent decisions and offering a prime opportunity to infuse projects with environmentally conscious considerations. Through comprehensive analyses, rigorous risk assessments, and thorough feasibility studies, practitioners pave the way for identifying sustainable pathways while optimising overall project efficiency. This planning not only establishes the project's goals but also determines its potential to stand as a beacon of sustainability.

Geotechnical engineers play a significant role in promoting sustainability during the planning stage through several key strategies (Basu et al., 2015):

Early Stakeholder Engagement: Engaging stakeholders at the outset of the planning process serves as a crucial step. This involves involving individuals or groups with vested interests in the project, such as landowners, residents, businesses, and government agencies. By soliciting their input early on, geotechnical engineers can ensure that their unique needs are addressed, averting potential conflicts and project delays.

Thoughtful Evaluation of Environmental Impacts: Geotechnical engineers must meticulously assess the environmental consequences of various options. This encompasses an in-depth consideration of diverse factors, including the use of different materials, construction methodologies, and maintenance practices. It is important to note that the most sustainable choice may not always be the most obvious, underscoring the necessity for a comprehensive evaluation of all available options. For instance, in their selection of more sustainable options, engineers must thoroughly consider all aspects from start to finish. Take 'precast retaining wall panels' as an example: they might appear as a sustainable choice if the precasting is done on-site or in a nearby factory. However, for a project where the prefabricated panels need to be transported from a distant location, the energy consumption during transportation might outweigh their sustainability benefits. Another illustration could be the use of certain types of foundation systems, which may have lower material requirements but higher installation energy costs, leading to a nuanced evaluation of their overall environmental impact.

Integration of Sustainable Materials and Practices: Geotechnical engineering offers a wealth of opportunities for integrating sustainable materials and practices. For example, incorporating recycled aggregates into concrete compositions and employing bio-based geopolymers as binders are potent avenues (Chindaprasirt, 2022). Additionally, the adoption of sustainable construction practices can substantially reduce waste generation and optimise energy utilisation. There are some challenges in using sustainable materials which are further discussed later in this paper.

By adhering to these best practices, geotechnical engineers significantly strengthen the sustainability aspect of geotechnical projects. This proactive approach not only aligns with contemporary environmental imperatives but also contributes to the long-term resilience and success of projects in a rapidly evolving global landscape.

3.2 DESIGNING FOR A GREENER FUTURE

Transitioning into the design stage, the endeavour to embed sustainability fuses creativity with pragmatic engineering. The central wisdom lies in crafting solutions that are not only functional but also sustainable, without compromising performance. Engineers navigate through an array of green materials and innovative techniques, selecting those that harmonise with ecological consciousness and structural efficacy. Design decisions carry the torch of sustainability, illuminating the path to a greener future.

Further to key strategies mentioned above for the planning stage, which are valid in the design stage, there are further key strategies that geotechnical engineers could adopt to result in more sustainable solutions (Basu et. al, 2015):

Design for longevity: Geotechnical practitioners can design projects to last longer using durable materials and construction methods. This can help to reduce the need for maintenance and repairs, which can have a positive environmental impact.

Design for resilience: Geotechnical practitioners can design projects to be more resilient to extreme events, such as floods, earthquakes, and storms. This can help to protect people and property from damage.

Design for adaptability: Geotechnical practitioners can design projects to be adaptable to changes in the environment or in the needs of the community. Design for adaptability is a design approach that aims to make a structure or system able to adapt to changes in its environment or use over time. This can be achieved by using flexible materials and components, designing for future expansion or modification and incorporating redundancy into the system. This can help to ensure the project remains sustainable over time (Vahidifard et al., 2021).

3.3 CONSTRUCTING SUSTAINABILITY

As the project embarks on the construction stage, the focus on sustainability remains unwavering It is here that practitioners take a multidimensional approach, considering methodologies, materials, energy consumption, and waste management. Carefully curated construction techniques, materials sourced with sustainability in mind, and strategies that minimise waste and toxicity collectively foster an environment of responsible construction. The construction stage becomes an arena where principles are translated into tangible results.

3.4 COMMISSIONING FOR THE LONG HAUL

Beyond completion, the torchbearers of sustainability shift their focus to the operation stage. The legacy of sustainable practices extends into this phase, where choices made during planning, design, and construction shape the ongoing operational efficiency of the project. Here, the emphasis rests on continual monitoring, maintenance, and optimisation to ensure that the project upholds its commitment to sustainability. Every operation becomes a testament to the far-reaching impact of thoughtful decisions taken earlier in the project lifecycle.

4 SUSTAINABLE MATERIAL SELECTION: GUIDING ENGINEERS TOWARDS A LOW CARBON FOOTPRINT

The use of sustainable materials in geotechnical engineering is a growing field, with many opportunities to reduce the environmental impact of geotechnical projects. However, there are also some challenges associated with using these materials, such as their lack of availability, high cost, and uncertainty about their long-term.

For instance, bio-based polymers, often lauded for their reduced environmental impact, may raise questions about their longevity and performance under varying conditions. Similarly, recycled plastics, while contributing to waste reduction, can prompt inquiries about their resilience in harsh environments. Additionally, the reduced use of coal as a fuel source, a sustainable practice, may lead to a limited future source of fly ash and slag for replacing Portland cement in concrete. These examples underscore the need for thorough assessments and ongoing monitoring to ensure sustainable materials maintain their environmental benefits over extended periods.

Despite these challenges, there are many reasons to use sustainable materials in geotechnical engineering. These materials can help to reduce the use of virgin materials, minimise waste, and conserve energy. They can also save money in the long run by reducing the need for maintenance and repairs, and by increasing the lifespan of structures. Additionally, sustainable materials can help to improve the quality of life for people and the environment by reducing pollution, creating jobs, and promoting sustainable development.

Some specific examples of sustainable materials that are being used in geotechnical engineering include recycled aggregates, bio-based geopolymers, and low-carbon concrete. As the demand for sustainable materials grows, the availability and cost of these materials is likely to decrease, and their performance and standardisation will improve. As a result, the use of sustainable materials in geotechnical engineering is likely to become more common in the future (Mills and Attoh-Okine, 2014).

4.1 CARBON FOOTPRINT HIERARCHY OF MATERIALS

One of the critical factors in achieving sustainability across the lifecycle of a project lies in the careful selection of materials. Each material employed in construction bears a unique carbon footprint, reflecting its environmental impact from production to disposal. It is imperative to increase awareness among engineers regarding this material hierarchy, aiding them to make conscientious choices that align with sustainability goals.

Figure 1 below illustrates the material hierarchy for typical materials utilised in the civil engineering project (WSP,2023), which underscores the varying carbon footprints associated with different materials. This hierarchy serves as a guiding principle for engineers, as they navigate the landscape of material selection.

Metals, such as steel and aluminium, stand at the higher end of the hierarchy, carrying significant carbon footprints due to energy-intensive extraction and processing. Plastic, another high-carbon material, joins this tier due to its petroleum-based origins and limited recyclability. Notably, concrete, despite its ubiquity in construction, holds a considerable carbon footprint, primarily owing to the energy-intensive cement production process.

Conversely, materials like earth fill and treated timber emerge as the flagbearers of low carbon footprints. This insight underscores the wisdom of harnessing natural resources that demand fewer energy inputs for extraction and processing.

To further diminish carbon footprints, the integration of recycled materials emerges as a transformative strategy. Utilising byproducts of industries, such as coal fly ash and iron furnace slag, as pozzolanic additives in concrete not only enhances its performance but also reduces the environmental impact by utilising materials that would otherwise go to waste.

In conclusion, the journey towards sustainability demands a keen understanding of material hierarchy. Engineers possess the power to sculpt a project's environmental legacy by making sensible material choices. This entails a conscious shift towards low carbon footprint materials, recycling, and judicious utilisation of industrial byproducts. As engineers learn this attitude into their practice, each project takes steps towards a future defined by sustainable ingenuity and responsible construction practices.

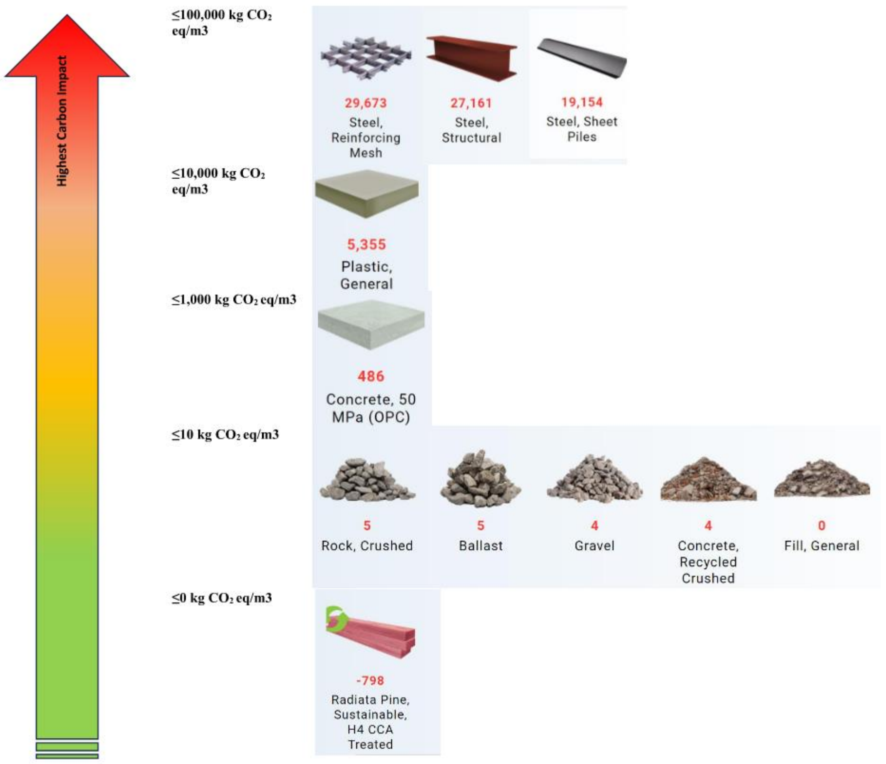

Figure 1: Material hierarchy in carbon impact

In the following sections of this paper, we will demonstrate how geotechnical engineering practices can contribute to sustainable infrastructure development in accordance with the IS Rating Scheme, using case studies from Australia as examples of successful implementation. These projects exemplify how sustainable geotechnical solutions can not only meet the requirements of the ISC but also help advance the broader goals of sustainability and resilience in civil engineering projects.

5 CASE STUDY 1: RESIDENTIAL COMPLEX DEVELOPMENT IN SYDNEY

5.1 PROJECT DESCRIPTION

A residential complex project was proposed on reclaimed land with underlying soft marine deposits in Sydney. This project encompasses four separate multi-storey residential buildings, each with one or two-level basements, along with a communal garden zone and associated civil works. The civil works include the construction of new access roads within the site, a subsurface drainage system, underground utilities, and landscaping efforts. The proposed construction has been planned in different stages, with Stage 1 involving the construction of Block D on the eastern end of the site, as well as three access roads, as highlighted in Figure 2.

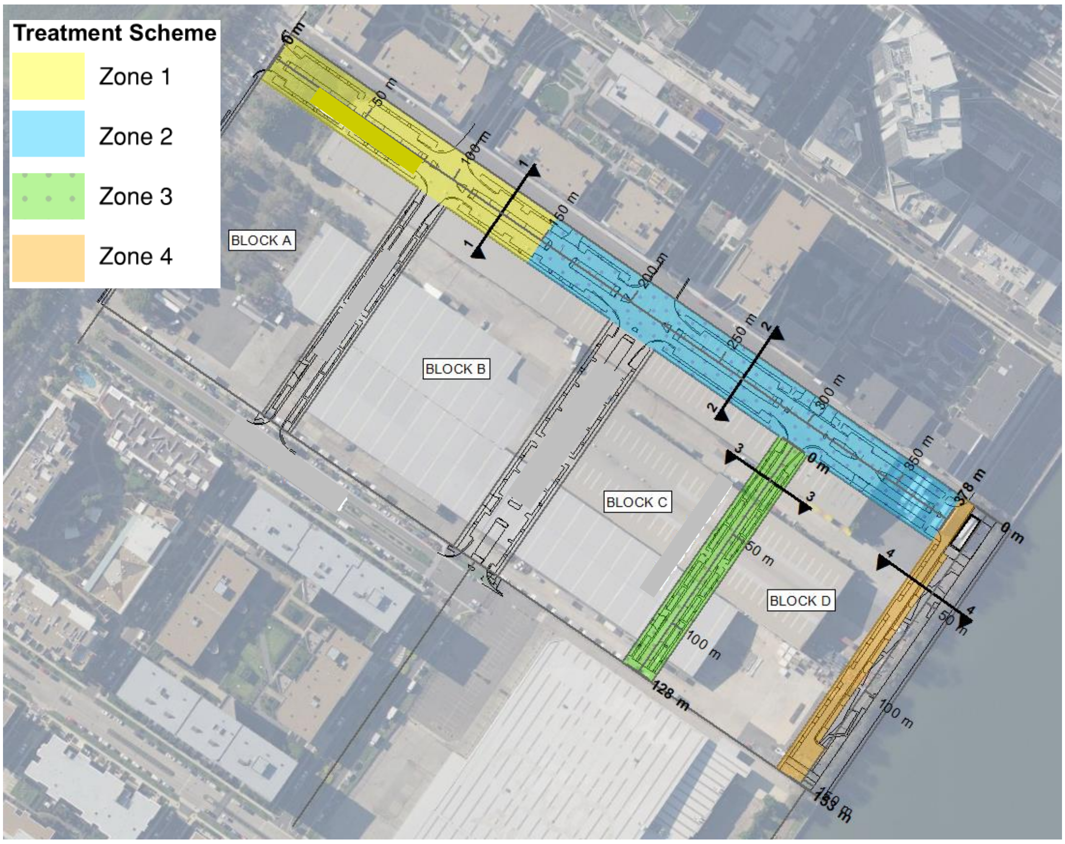

Figure 2: Proposed Stage 1 access roads

Available geotechnical data indicated that the site was underlain by compressible materials comprising a variable thickness of very soft to firm clay/peat (estuarine, alluvial, and/or dredged fill) and loose to medium-dense sand. The upper few meters of the soft soil were inferred to be associated with the reclamation and filling works undertaken on site between the 1930s and 1950s (Figure 3). Further information about the site geology and history has been presented by Johnston and Terzaghi (2021). The site was occupied with warehouse buildings used for the storage of goods. It has a history of ongoing settlement issues, with some warehouse structures and access roads across the site having undergone excessive settlements, resulting in cracks in internal partitions, floor slabs, and tension in the steel frameworks. These issues warranted repair and rectification works.

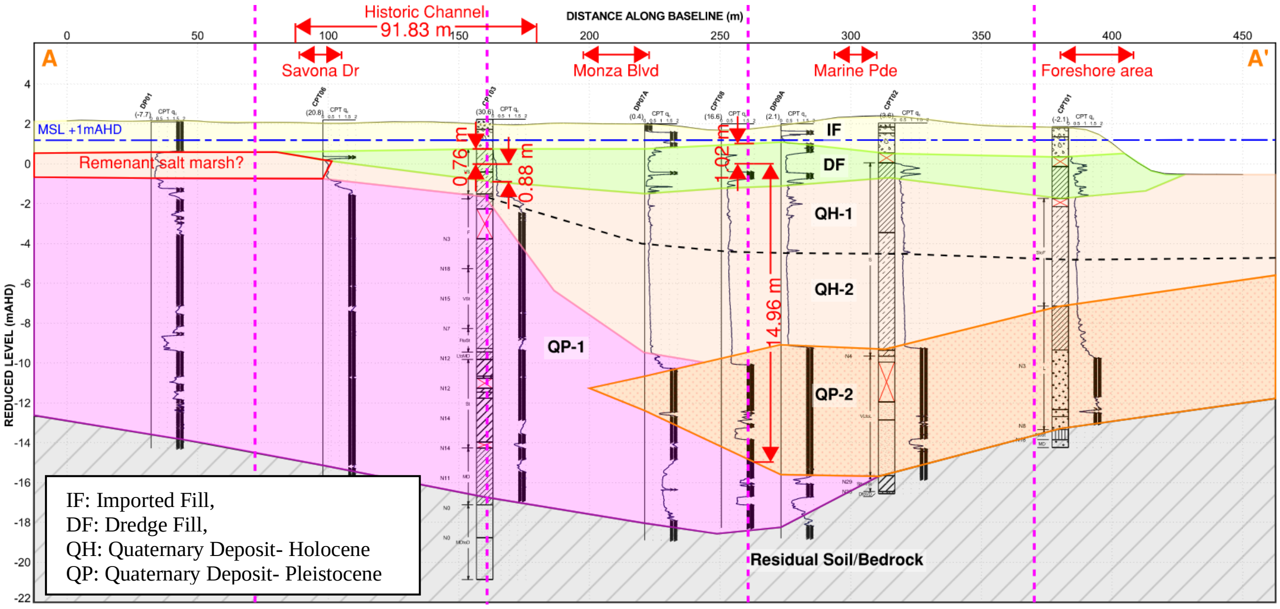

Figure 3: Inferred subsurface profile at the project site.

5.2 SUSTAINABILITY CONSIDERATIONS

Planning Stage

The concept design by another consultant considered a structural solution, including suspended slabs founded on bored piles that extended into the sandstone bedrock along all access roads (see Figure 4). This approach aimed to mitigate the impact of anticipated long-term consolidation and creep settlements. It was influenced by the solution adopted in neighbouring developments and concerns regarding the long-term settlement of the proposed access road. There was also apprehension about the risk of lateral squeeze and its potential adverse effects on the foundations of neighbouring properties and an adjacent box culvert with piled foundations. The available geotechnical investigations had been planned and executed with this in mind, focusing primarily on confirming the top of the bedrock and characterising the sandstone. As a result, they provided very limited data on the underlying soft soil layers.

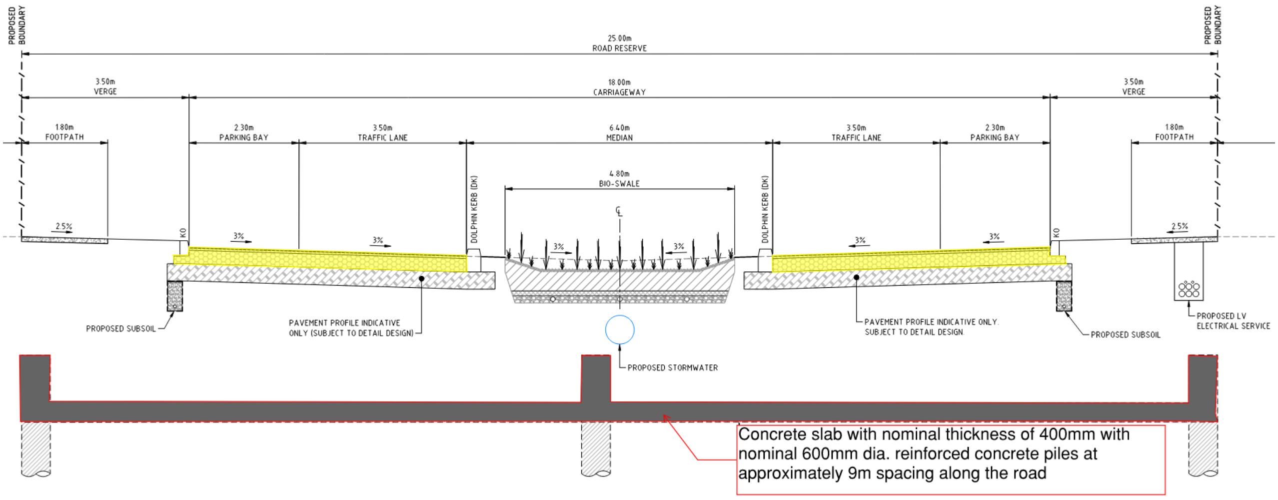

Figure 4: Typical section of piled slab solution proposed by others

A due diligence study by the authors considered other feasible, more sustainable, and cost-effective solutions, such as preload/surcharge and rigid inclusions, instead of the extensive piled option. Following a subsequent desktop study on the site's history and geology (refer to Johnstone and Terzaghi, 2021), as well as an advanced site investigation, an opportunity emerged to employ preload and surcharge ground improvement methods in lieu of extensive piled slabs for the proposed concept design. A site walkover around the site and neighbouring developments was conducted, which highlighted potential issues associated with a piled option. This included cracks in civil and landscape structures, along with differential settlements observed at the interface of the piled structures, access roads, pathways, stairways, and other areas integrated into the landscape that were founded on natural untreated ground. This also underscored the advantages of preloading the site, further affirming the merit of this soft option compared to the initially proposed rigid piled option.

Design Stage

A comprehensive geotechnical investigation was meticulously planned and executed. This included boreholes, Seismic Piezocone Penetration Tests (SCPTu) and Seismic Dilatometer Marchetti Tests (SDMT), as well as laboratory testing on undisturbed samples retrieved from the underlying soft soils. The information garnered, coupled with desktop studies on the site's history, geology, and anecdotal data regarding previous settlements on site, enabled us to accurately characterise the variable subsurface profiles across the site. This exercise applied advanced knowledge in soft soil engineering beyond standard engineering practices.

Finite Element Modelling using Plaxis was employed to assess potential long-term settlements at the proposed road embankments and their impact on existing structures and utilities. The soft soil creep material models utilised in Plaxis were extensively calibrated against available information on site loading and settlement history. Ground improvement strategies for the proposed earthworks were designed, incorporating preload and surcharge methods, as outlined in Table 1 (refer Figure 2 above for the extent of treatment zones on the site plan).

Table 1: Summary of analysis results

Treatment Zone	Treatment Scheme	Preload Duration	Residual Settlement After Removal (Note 1)
Zone 1	Preload	6 months	15-20mm
Zone 2	Preload with 0.5m surcharge	6 months	40-50mm
Zone 3	Preload with 0.5m surcharge and 2.0m spaced wick drains	9 months	30-50mm
Zone 4	Preload with 0.5m surcharge	9 months	40-50mm

Note: 1. Variable settlement anticipated in each zone due to variable thickness and potentially different stress history of soft soil layer at each zone.

A geotechnical instrumentation and monitoring plan was developed and implemented during the construction phase. This plan encompassed settlement plates and extensometers placed strategically across the fill embankments, along with inclinometers positioned near sensitive structures and utilities, as well as along the site boundaries. The results of this monitoring closely aligned with the predicted outcomes. This alignment signifies a well-founded characterisation of the site's subsurface condition and validates the efficiency of the selected approach. This correlation between actual and anticipated outcomes provided confidence in the accuracy of the subsurface assessment and the appropriateness of the chosen methodology.

Construction Stage

Earthwork specifications are crucial documents that outline the precise requirements for earthwork activities, encompassing excavation, fill, material properties and compaction. They play an instrumental role in ensuring construction safety, durability, and performance of earthworks, while upholding necessary standards. Nevertheless, challenges often emerge during their development.

One prominent issue arises from blindly copying specifications from unrelated sources or amalgamating more stringent requirements from literature or other projects without considering their relevance to the specific site. This can lead to the inclusion of unnecessary or impractical requirements for the project, resulting in needless costs, complications during implementation, and potentially less sustainable solutions. Moreover, specifications may stipulate materials without a comprehensive assessment of their impact and associated expenses. This can inadvertently lead to the selection of costlier or harder-to-acquire materials, overlooking more feasible and sustainable alternatives.

Furthermore, specifications may occasionally overlook site-specific constraints and opportunities for value engineering. For instance, a specification might insist on extensive excavation without considering the potential for reusing excavated material elsewhere on the site. These oversights can lead to inefficiencies and missed opportunities for cost savings or reducing the project's carbon footprint.

To tackle these challenges, a collaborative approach is imperative. Involving all stakeholders, including the geotechnical engineer, contractor, and owner, ensures that specifications are tailored to the project's unique needs. Regular reviews are essential to maintain their relevance and feasibility.

Additional strategies for effective specification development include clarity and conciseness, adherence to industry standards and codes, flexibility to accommodate evolving project needs, and seeking feedback from contractors who bring valuable practical insights.

By adopting these practices, earthwork specifications can be crafted to not only meet the necessary standards but also streamline operations, control costs, and enhance overall project efficiency and performance.

In this project, a project-specific earthwork specification was devised, considering the site-won material characteristics and performance requirements for fill to be used in different zones of the proposed earthworks (e.g., fill within the zone of influence of pavement, pathway subgrade, structural backfill, swale, or garden beds), as well as project constraints. Collaborative efforts between the designer, contractor, and developer allowed us to develop an earthwork specification and design documents that enabled the reuse of most site-won material on site, with or without treatment, without compromising the performance requirements.

5.3 SUSTAINABILITY OUTCOMES

The proposed ground improvement, utilising preload and surcharge methodology, led to saving of over 7000 m^3 of concrete and more than 800 tons of reinforcement steel in this project, as opposed to the initially considered pile slab solution. This not only resulted in a substantial cost saving but also a significant reduction in carbon emissions - a reduction of over 5500 tons. It is worth noting that this figure does not account for any potential reduction in carbon emissions due to decreased imported fill volume and re-use of demolition and excavation materials on site.

6 CASE STUDY 2: MAJOR HIGHWAY PROJECT

6.1 PROJECT DESCRIPTION

The other example case was a new highway project, which its details cannot be disclosed. The scope of works comprised major cut and fill, several bridge construction and tunnelling works. Value engineering prompted a review of concept design bridge foundations, retention systems, and site investigation results to seek alternative structural solutions and earthworks specifications to make the project commercially viable.

6.2 SUSTAINABILITY CONSIDERATIONS

The following are some of the sustainability considerations that were made in the reference design of this project:

- **Use of more sustainable structural alternatives:** The project team considered a variety of structural alternatives, including those that used recycled materials or that were designed to be more energy efficient. The team selected a structural solution that was both cost-effective and sustainable. For example, some of the bridge structure systems were changed and optimised and concrete retaining walls were replaced with soil nail walls.
- **Reducing the amount of rock that needed to be processed:** The initial cost estimate of the project assumed that all the excavated rock from the tunnelling works would need to be processed into earth fill. However, a literature review was undertaken on the specifications of earthworks used globally and on the rearrangement of rock fill particles as well as subsequent settlements. Based on this research, the team concluded that maximum particle size of rock fill materials could be revised in the specification so some of the site-won rock could be used in lower portions of high fill embankments or noise mounds without being processed. This reduced the amount of rock that needed to be processed, which saved energy and reduced the environmental impact of the project.
- **Providing opportunities for innovation:** The road authority provided opportunities for the tenders to propose and use more innovative solutions. This helped to ensure that the most sustainable solutions were implemented.

6.3 SUSTAINABILITY OUTCOMES

The exemplary approach taken in the value engineering and reference design of this project not only ensured its financial viability but also led to substantial sustainability benefits. These outcomes were achieved through a combination of strategic decisions and innovative practices, highlighting how the project aligns with and contributes to broader sustainability goals.

- **Reduction in material processing for lower energy consumption**

The decision to utilise site won rock without extensive processing was a pivotal sustainability move. It substantially curtailed the energy-intensive processes required for material crushing, grinding, and other forms of processing. This reduction in energy consumption not only would lower the project's carbon footprint but also minimise its environmental impact, aligning with global efforts to conserve energy and reduce emissions.

- **Local and site won material utilisation for reduced traffic movements**

By prioritising the use of locally sourced materials and use of site-won materials, the project would significantly reduce the need for long-distance transportation of construction materials. This not only curtails associated carbon emissions from transportation but also bolsters the local economy by supporting nearby material suppliers.

- **Innovation: departure from standard procedures**

The project's willingness to depart from conventional practices and consider alternative solutions demonstrated a commitment to innovation. This mindset not only would lead to more sustainable outcomes but also set a valuable precedent for future projects. By fostering an environment of creativity and open-mindedness, the project would contribute to a culture of continual improvement and adaptation within the industry, thereby advancing the cause of sustainability.

7 CONCLUSIONS

In an era defined by environmental challenges, integrating sustainability into civil engineering is paramount. Projects' long-lasting impact demands a commitment to a resilient, eco-conscious future. This includes minimising carbon emissions throughout a project's life cycle, a critical step in combating climate change.

Geotechnical engineering plays a vital role in this pursuit. Aligning with global and local sustainability goals, it exemplifies practices like the Infrastructure Sustainability Rating scheme. Through strategic planning, thoughtful design, and conscientious construction, geotechnical engineers lead the way.

Hierarchy of materials in terms of carbon footprint, importance of project specific specifications thoughtfully developed and tailored for a project were discussed and elaborated in this paper.

Case studies presented which highlighted these principles in action and it was demonstrated that such considerations during planning, design and construction phase of a project could result in substantial cost savings and emissions reduction. Earthwork specifications tailored to site-specific conditions further enhanced sustainability.

In essence, geotechnical engineering embodies innovation, environmental consciousness, and responsible construction. Practitioners are the guardians of a greener future, where engineering aligns with ecological wisdom.

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INVESTIGATION OF A CALCINATED CLAY PRODUCT FOR SHOTCRETE SUPPORT

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ABSTRACT

This paper presents research results for three shotcrete mixes that include up to 50% cement replacement by calcinated clay. The results are part of a multi company research project under way at the WA School of Mines and show that a 25-30% saving can be achieved. In addition, a large amount of CO_2 reduction (up to 40%) can be realized by replacing the cement within a conventional shotcrete mix. Additionally, large gains in early strength can lead to higher productivity. Also, the long-term strength is significant, and the failure mechanism is compatible to soft response to violent loading, likely to minimize shotcrete ejection.

1 INTRODUCTION

Shotcreting is a ground support technique in which a specially designed concrete mix is sprayed at high speed, onto rock excavation surfaces to improve rock mass integrity and assist load carrying capacity at the rock surface. Shotcrete minimizes time-dependent rock deformation, slabbing and violent rock ejection of highly stressed rock, thus improving the safety of the underground mining excavations, where equipment and mining personnel are exposed. In Australia, the use of shotcrete has continued to grow in the mining industry since the late 1980s due to its success in stabilising excavations driven in difficult conditions. The future use of shotcrete will continue and is likely to increase further as mines attempt development within the higher stress regimes and more difficult conditions that generally accompany mining at depth (Villaescusa et al., 2023).

A typical mining tunnel development (5m W by 5m H) advances 3.5 m at a time with 5-7 m^3 of sprayed concrete used to achieve a 50-75 mm thick surface support layer. Thus, for a typical wet mix, the cement component of a 1 m^3 of shotcrete ranges from 400-450 kg of cement. Additionally, for a typical underground mine, the usage of cement from shotcreting activities typically ranges from 2,000 to 20,000 t/year, depending upon the mining method.

In addition to material and transport cost of the cement to the remote mine site locations in Australia, an excessive amount of CO_2 is being emitted during the production of clinker, the main component of cement. This is due to burning of the limestone ($CaCO_3$) during cement production. Historically, to limit the emissions, the cement manufacturing process has improved the fuel efficiency of the burning process and included supplementary cementitious materials such as silica fume and fly ash. Nevertheless, 60% of the emissions are due to the use of limestone. Recently, a sustained effort to quantify and reduce CO_2 emission has started within Australia at large and the mining industry in particular. Consequently, the Western Australian School of Mines (WASM) has started a multi-company research project to investigate the replacement of a significant portion of the cement being used in the mining industry. The cement is being replaced by calcinated (activated) clay which is a low cost (and abundant), supplementary cementitious material.

2 CLAYSTONE DEPOSIT

The clay material being investigated is sourced from a deposit located within the late Archaean Eastern Goldfields Superterrane (EGS) of the Yilgarn Craton, Western Australia (Ambrose, 2023). The Kalgoorlie Clay (KalClay) Project mining and exploration leases are located adjacent to the sealed Yarri Road, approximately 10 km northeast of the city of Kalgoorlie and 7 km southwest of the Kanowna Belle Gold Mine (Figure 1).

Within the project area deep weathering and the chemical characteristics of the underlying sedimentary units have contributed to the significant thickness (average 50 m) of a clay layer. The underlying stratigraphy is dominated by felsic-derived immature sediments, with most exploration holes ending in very fine-grained siltstones and mudstones. The exploration drilling to date has shown an abundance (footprint 8 km by 2 km) of clay mineralisation probably associated with weathering of the Black Flag Beds. Overall, very thin, typically less than 1 m thick, young, transported sediments cover the greenstone stratigraphy. The underlying sedimentary-dominated stratigraphy is extremely weathered with the base of complete oxidation generally occurring at 45 – 65m depth in all holes. The clays comprise predominantly silica, kaolin and mica (interpreted in XRD as muscovite). A typical chemical analysis of samples obtained above the base of complete oxidation is shown in Table 1.

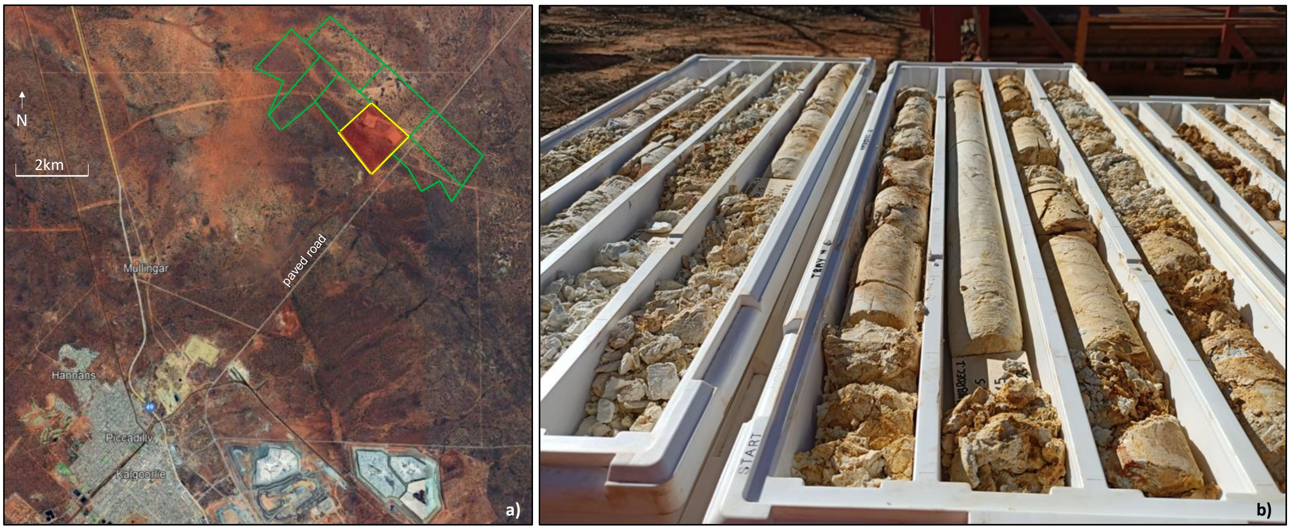

Figure 1: Details of KalClay project showing a) mining (yellow) and exploration leases with respect to Kalgoorlie and b) typical claystone drilling results

Table 1: Chemical analysis of the KalClay material

Material	**Composition**	**%**
Mica	$X_2Y_{4-6}Z_8O_{20}(OH,F)_4$	23
Silica	SiO_2	52
Kaolinite	$Al_2Si_2O_5(OH)_4$	23
Halite	NaCl	<1
Rutile	TiO_2	1

3 SHOTCRETE SUPPORT

Two conflicting requirements can be identified for a shotcrete mix. Firstly, it must have the rheological properties of a fluid to be pumped and sprayed. Secondly, it must have the mechanical properties of a solid to create a stabilising structural layer. The rheology of the mix depends on the fluid/solid constituents, their particle size distribution which in turn affect the mechanical properties of the in-situ paste, its hardening, and the mechanical properties of the final hardened layer.

An infinite variety of possible loadings and equally large numbers of possible boundary configurations exist. However, two main mechanisms are considered here, firstly a stress-controlled deformation mechanism and secondly, a structurally controlled deformation mechanism. The total response and total capacity can be a combined response mode comprising a limited number of fundamental responses including:

- Compression
- Tension

- Shear
- Torsion
- Flexural
- Combinations

The typical response modes are illustrated in Figures 2 with examples in Figure 3. Each fundamental response mode has an associated capacity (in terms of force and displacement) or mechanical properties such as strength (in terms of compressive, tension, shear and flexure) in elastic and plastic phases. The mechanical properties can be determined from different laboratory test arrangements.

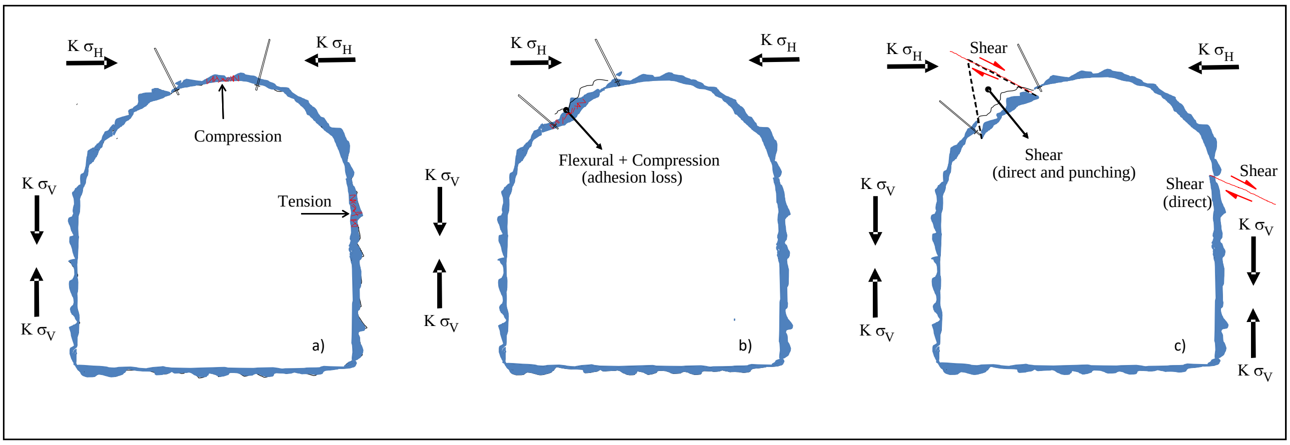

Figure 2: Modes of response for excavation with filled roughness and covered to a minimum shotcrete thickness a) Compression and tension, b) Flexural and compression and c) Shear modes of failure (Modified after Windsor, 1999)

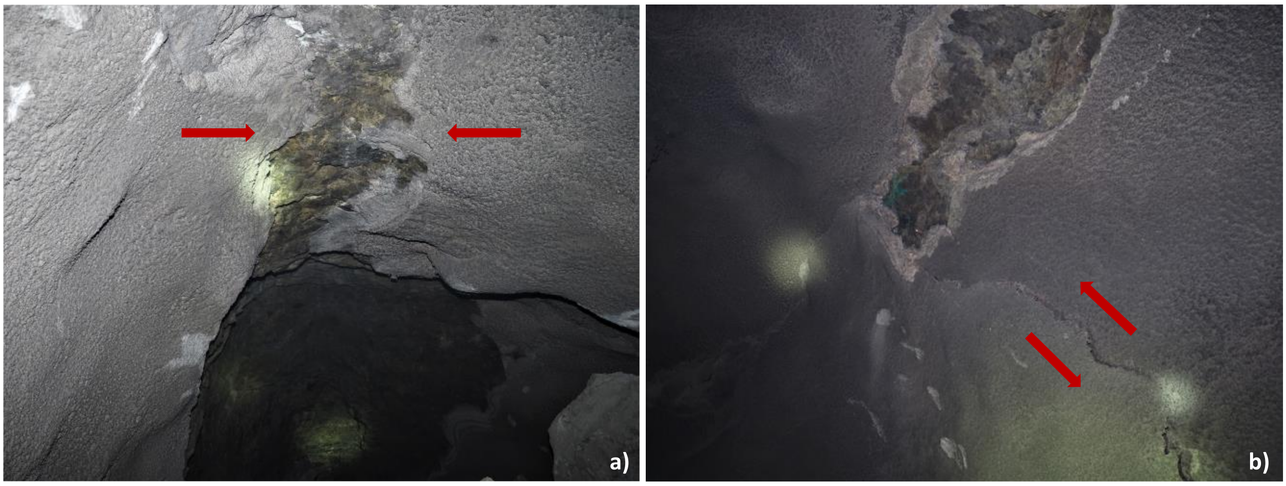

Figure 3: Examples of shotcrete failure a) compression and b) shear and punching

3.1 SHEAR STRENGTH OF FRESHLY SPRAYED SHOTCRETE

Prior to hardening, the mechanical properties of the paste are dictated by the cementitious matrix comprising the cement, mineral additives, chemical admixtures, and the water. After hydration, the shotcrete should possess the mechanical properties of the hardened matrix plus some additional strength due to the presence of coarse aggregate particles, mesh reinforcement and fibres. It is important to note that these mechanical properties improve with hydration from those of the wet paste, to the stiff paste, to the hardened paste and finally to the fully hardened and cured shotcrete. Consequently, the mechanical properties of freshly sprayed shotcrete are those associated with the cementitious matrix. Therefore, it is predominantly the changes with time of the curing shotcrete paste that will indicate the mechanical response of freshly sprayed shotcrete.

The yield strength of the fresh shotcrete pastes for the research described here was determined using a purposely constructed and calibrated vane shear test apparatus (Figure 4). During the mixing period the hydration products in the shotcrete pastes are in the fluid gel state. After approximately 1.5 to 2 hours curing time, the hydration products start transforming into a solid gel state and its shear strength can then be readily determined using conventional uniaxial or triaxial compression test methodologies.

Figure 4: A large-scale viscometer used to determine yield stress of full shotcrete mixes

Two structural requirements can be identified for a freshly sprayed shotcrete layer. Firstly, it must support its own mass within minutes of being applied to the surface and, secondly, it must support the superimposed mass of an estimated unstable volume of rock. In the first instance, the shotcrete supports its own mass by development of an adhesive bond strength (comprising adhesion and mechanical interlock) between itself and the substrate and by development of intrinsic shear strength.

The minimum shear strength required for shotcrete to support its own weight is typically about 4 Kpa (Villaescusa et al., 2013). In almost all cases, where bond and shear strength develop simultaneously after spraying, both laboratory investigations and in situ experience have shown that the required strength levels for shotcrete to support itself are easily achieved.

During service, the shotcrete must be capable of supporting the mass of loose rock blocks that may become unstable and represent a risk to personnel that enter an excavation. The specific arrangement of excavation span, stress and structural geology associated with each excavation will be different and the specification of unstable volume of rock are linked to the potential block shapes that can form due to the geological discontinuity array at a particular location. The computer program SAFEX can be used to show that within a few hours of spraying, shotcrete is quite capable of supporting a significant volume of unstable rock. Figure 5 shows a 5 m y 5m tunnel back with a potential 1.5-ton block shape (0.75 m apex). The calculations show that block can be stabilized by having a 50 mm thick shotcrete layer that has developed a 100 Kpa shear strength. That is, it can be noted that for Block Number 6, the Stability Index (safety factor) increases from 0.44 to 1.0 with the addition of the shotcrete layer. Similarly, each potential block geometry having a Stability Index less than 1 needs to be analysed separately to determine if its occurrence is likely in terms of mass, depth of failure and tunnel profile construction.

Saw (2015) indicated that freshly sprayed shotcrete stability can be determined using a known volume or mass of unstable rock in conjunction with the layer thickness and time after spraying. Saw (2015) provided examples in which a 2.7 mass of instability (i.e., a 1 metre cube block) would be supported by 50 mm of shotcrete having developed a 97 KPa shear strength at approximately 3 hours and 50 minutes after spraying, for a conventional shotcrete mix with accelerator, synthetic fibres and aggregates. It must be noted that the volume of loose rock that may become unstable is naturally minimised during blasting by waves that vibrate the excavation surfaces and by subsequent mechanical or hydro-scaling procedures that clean the excavation surfaces. Consequently, a 100 KPa shear strength is usually sufficient for safe re-entry time in most cases of slab type instability.

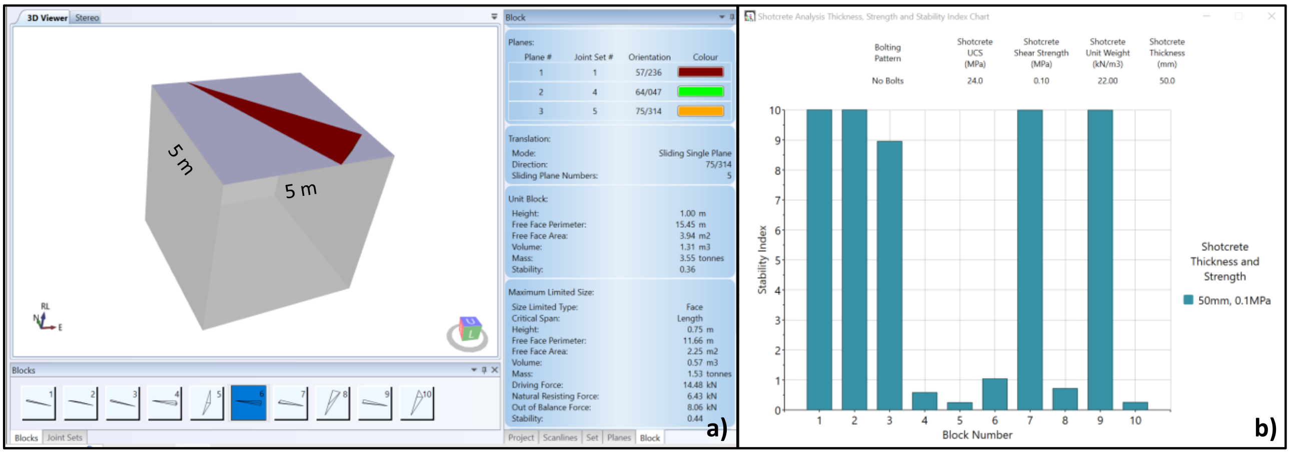

Figure 5: Example of shotcrete design for a) potentially unstable block geometry and b) stabilized using a shotcrete layer

3.2 PERFORMANCE OF CURED SHOTCRETE

The physical properties of the hardened layer that are important are its density, void ratio and permeability. However, the mechanical properties of a shotcrete layer that are most significant in rock support action include its strength (compressive, tensile, shear and adhesion) and its stiffness (flexural, biaxial and shear). These mechanical properties are dictated by the rheology of the paste, the hardening mechanism of hydration and the underground environmental conditions (i.e. temperature and moisture) during hydration and curing.

The SAFEX program can be used to show that a modest amount of shear strength is sufficient to stabilize most potentially unstable blocks (Figure 6). The calculation (for the same block geometry shown in Figure 5) shows that a block of nearly 3.5 tonnes and having 1.5 m apex height, exposed at a 5 m tunnel back, can easily be stabilized using a 50 mm thick layer of shotcrete having 1.2 MPa shear strength. For a conventional bolting pattern (1-1.2m by 1-1.2m), such unstable weight would be at the upper limit of possible unstable geometries (within the bolting pattern). That is, when an instability becomes significant, it is likely to first mobilize the reinforcement elements (Villaescusa et al., 2023).

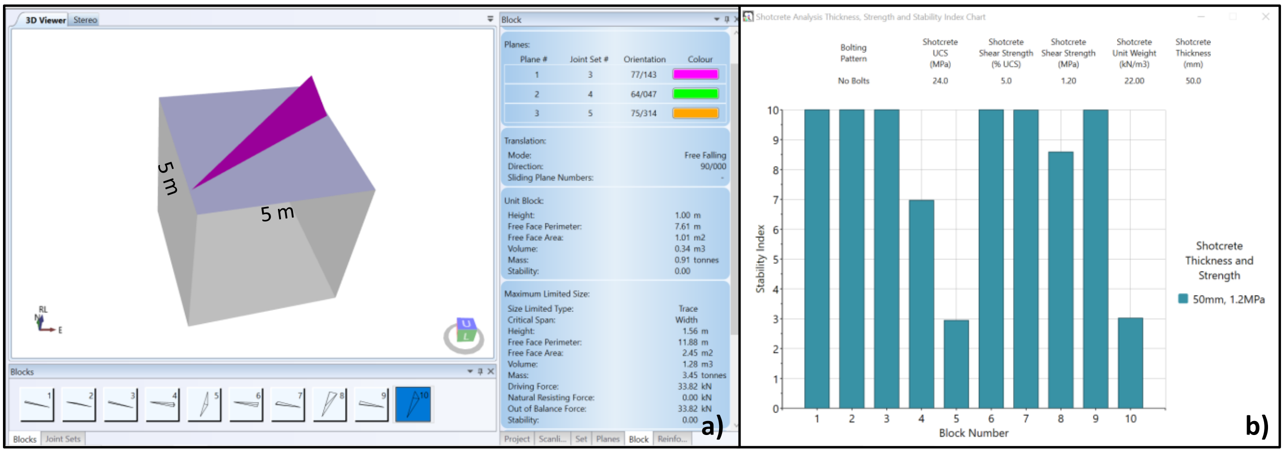

Figure 6: Example of a) potentially unstable block geometry b) stabilized with a modest amount of shear strength development

4 REPLACEMENTS OF CEMENT FOR IGO LIMITED SHOTCRETE MIXES

The WASM research to date has considered shotcrete mixes that include up to 50% cement replacement by the KalClay product. The variables studied to date include sand and aggregate grading, the water/solids, the chemical admixtures (retarder, water reducer and accelerant), the yield stress and the resulting slump. Some of the results are as follows.

Aggregate is the granular material, such as sand, crushed stone and gravel used with a cementing medium to form hydraulic-cement concrete or shotcrete (ASTM C125 – 10a). It occupies at least three quarters of the total volume of a given shotcrete mix and thus, its properties greatly affect the properties of the fluid and hardened concrete or shotcrete. Figure 7 shows the particle size distribution for three IGO Mine sites including a comparison with the typical combined grading used in Australia (Villaescusa et al., 2023). A cross section view of a typical 100 m diameter specimen can be seen in Figure 8 showing the effects of each particle size distribution.

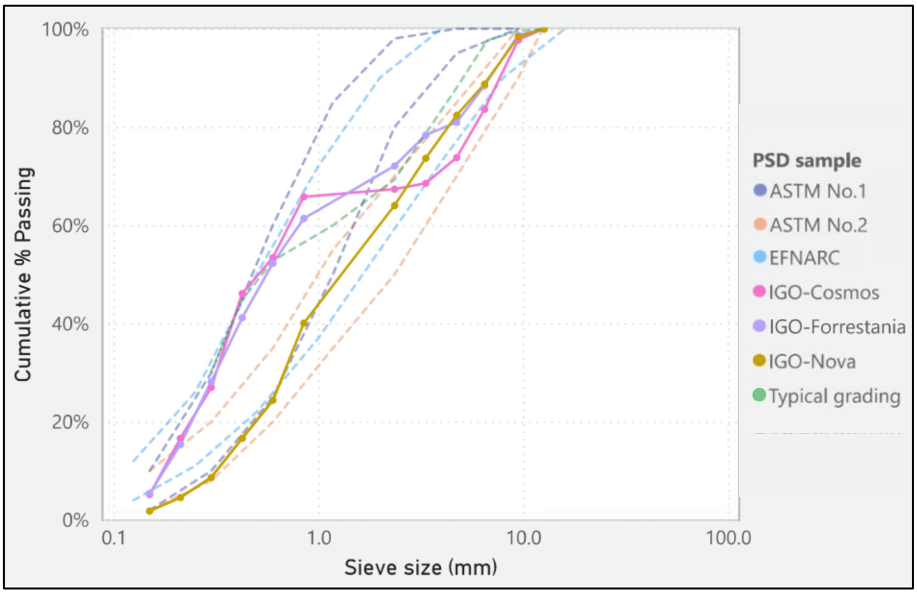

Figure 7: Grading limits for three IGO shotcrete mixes

Figure 8: Cross section of view of UCS specimen following testing for a) Cosmos, b) Forrestania and c) Nova Mines

Figure 9 shows the early strength development for several mixes for three mine sites at IGO Limited. The data shows that for mixes having 50% of the cement replaced by the KalClay material, the 100 Kpa shear strength capacity is achieved in less than 2 hours. Comparatively, the conventional mixes can not reach the 100 Kpa shear strength within the first 4 hours of curing time.

Figure 10 shows a plot of long-term strength for the conventional mixes used at several of the IGO Limited operations. These results can be compared with the data shown in Figure 11, where 50% of the cement component within the mix has been replaced by the KalClay product. The data indicates that the cement-KalClay mixes produce a more uniform strength development of sufficient engineering capacity. That is, the block theory calculations using SAFEX indicates that the cement-KalClay mixes develop sufficient shear strength to stabilize any block formed with a typical bolting pattern. Also, the failure mode of cement-KalClay is more plastic, which is important if shotcrete ejection or large time dependent deformations are likely to occur under high stress (Villaescusa et al., 2023).

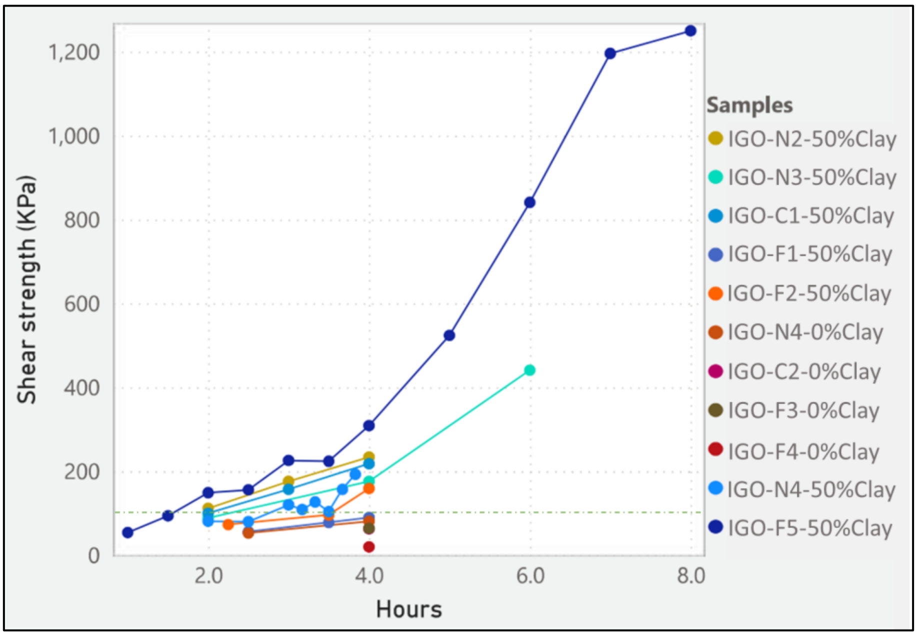

Figure 9: Early strength of shotcrete for several mixes at IGO Limited

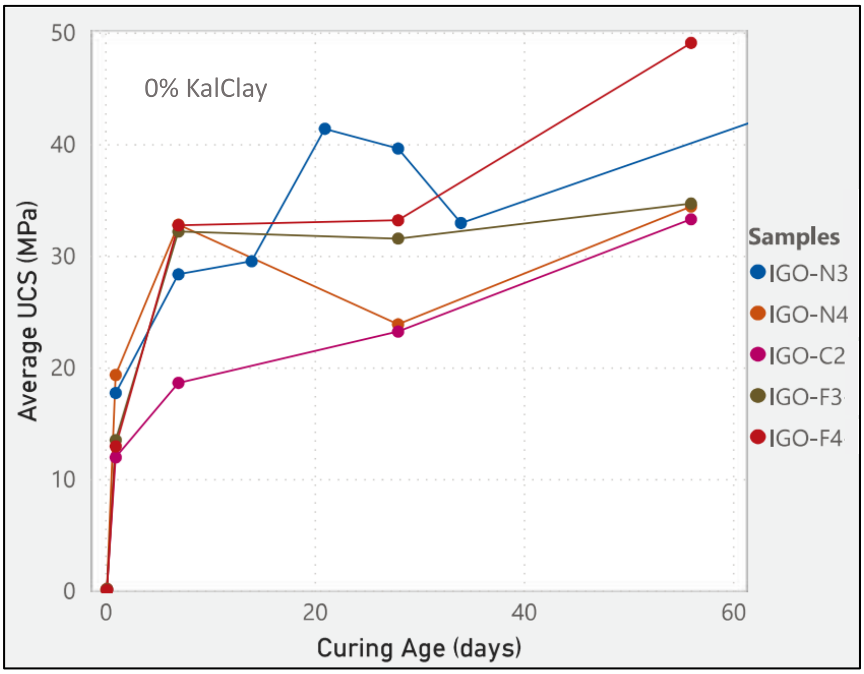

Figure 10: Long term strength of shotcrete for conventional mixes at IGO Limited

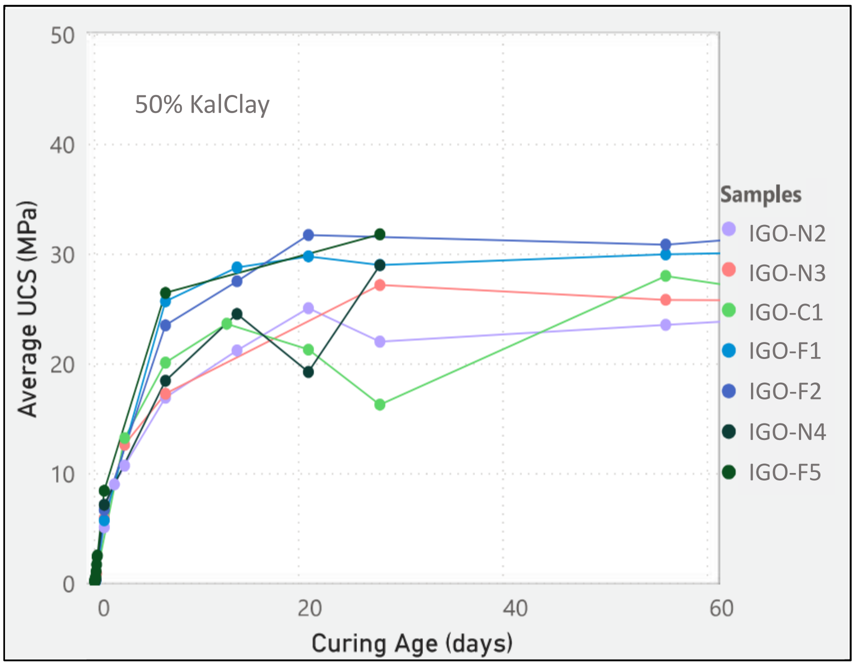

Figure 11: Long term strength of shotcrete for cement-KalClay mixes at IGO Limited

6 CONCLUDING REMARKS

The research results presented here indicate that a significant proportion of the cement component of a shotcrete mix can be replaced by the KalClay material. The percentage of replacement can vary according to the expected rock mass demand of early and long-term strength for a particular shotcreting application. The cost saving for a 50% replacement is estimated in the range of 25-30% with a 40% CO_2 emission reduction.

Also, for a similar curing age, the early strength for a shotcrete mix having equal proportions of cement-KalClay is greater than a conventional mix using cement only. This presents an opportunity for productivity improvement due to a decreased re-entry time. The re-entry time can be safely engineered using block theory calculations that rely on shear strength.

The long-term strength for a shotcrete mix having equal proportions of cement-KalClay provides sufficient shear strength capacity for ground support schemes that incorporate a typical rock bolting pattern. In addition, the strength development is uniform, and the failure mechanism is a soft, non-violent failure with much less micro-seismicity, as shown in Figure 12. This is important to minimize shotcrete ejection during large violent rupture following a seismic event while mining under high stress.

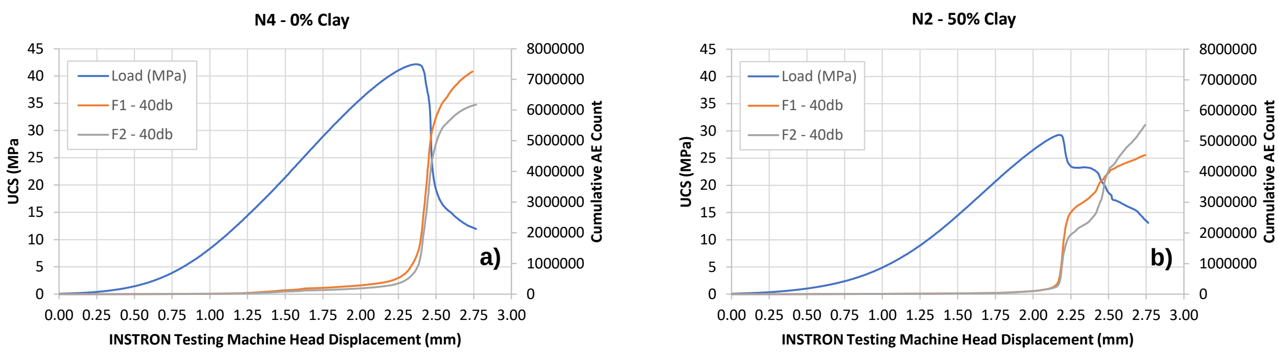

Figure 12: Micro-seismic response during sample loading a) conventional cement mix and b) 50% KalClay component

7 ACKNOWLEDGEMENTS

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EXPLORING THE IMPACTS OF ABUNDANTLY AVAILABLE SUSTAINABLE BY-PRODUCT MATERIALS IN AUSTRALIA ON STABILIZING EXPANSIVE SOILS

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ABSTRACT

This paper aims to examine the effects of utilizing readily available sustainable by-product materials in Australia for the purpose of stabilizing expansive soils. Some waste by-products, commonly found in Australia that can be employed for soil stabilisation are cement kiln dust, blast furnace slag, quarry dust, bagasse ash and fibre, rice husk ash, fly ash and bottom ash.

With support of industry a number of materials have been selected for characterisation. Extensive experimental tests utilizing bagasse fibre, bagasse ash, bottom ash, fly ash, and eggshell powder have been conducted at the University of Technology Sydney (UTS) to enhance the engineering properties of expansive soils. These tests have been supplemented by microstructural tests, numerical analysis, and comprehensive discussions. These pozzolanic materials are characterized by significant levels of calcium carbonate, silica, and alumina. Numerous tests have been performed using these by-products to investigate the impact of their composition in conjunction with lime or cement, the curing time, the particle size, the optimal blending ratios, on both treated and untreated soil properties.

Based on research and laboratory investigations, sustainable by-product materials have demonstrated substantial potential for enhanced durability, cost savings and long-term environmental benefits, compared to traditional cementitious agents in treating expansive soils. These materials offer improved soil strength, reduced swelling potential, enhanced soil ductility, and controlled deformation over time. However, the implementation of these sustainable materials in practice is not yet widespread among construction companies and road authorities in Australia. This paper addresses this concern and provides practical recommendations for adoption of these sustainable by-products in weak subgrade of roads.

1 INTRODUCTION

Soils containing expansive minerals can undergo volume changes, either expansion or contraction, based on variations in the availability of free water for absorption or evaporation. This phenomenon is referred to as swelling and shrinking, respectively. Expansive soils refer to fine-grained soil or weathered rocks that undergo significant volume fluctuations when exposed to changes in moisture content (Nelson and Nelson, 2013; Dang and Khabbaz, 2018). This swelling and shrinkage behavior typically occurs near the ground surface, where it is directly influenced by seasonal and environmental fluctuations. According to Fityus and Buzzi (2008) expansive soils are often unsaturated and contain clay minerals of the smectite group (Table 1). The extent of damage associated with expansive soils primarily depends on the level of monovalent cations absorbed by these clay minerals as expressed by Dang et al., 2017).

Table 1: Main expansive clay minerals (modified after Mitchell, 2001)

Expansive Clay (Smectite Group)	**Swelling Capacity** (Greater than original volume)	**Clay Activity** $A_c = \frac{PI}{Clay\ content(\%)}$
Bentonite (Sodium)	Up to 15 times	3 - 7.2
Montmorillonite (Sodium)	Up to 10 times	2.5 - 7
Bentonite (Calcium)	Up to 2 times	1.6 in average
Montmorillonite (Calcium)	Up to 2 times	1.5 in average

The distinction between expansive and non-expansive soils lacks a precise definition. Several guidelines are available for defining expansive soil based on its physical and chemical properties (e.g., AASHTO T 258-81, 2018; Australian Standard AS 2870, 2011; ASTM D4546-21). Some of the most common criteria for high potential movement include:

- Linear shrinkage (LS): Expansive soils typically exhibit a linear shrinkage exceeding 18%.
- Liquid limit (LL): Expansive soils typically have a liquid limit greater than 60%.
- Plasticity index (PI): Expansive soils usually have a PI higher than 35%.
- Soil activity (Ac): Expansive soils tend to have a soil activity exceeding 1.25.
- Soil suction: Expansive soils generally possess a natural soil suction at the time of construction exceeding 380 kPa (this is not a general rule for all expansive soils and depends on clayey soil type, natural moisture content, and climate conditions).
- Swell pressure: According to ASTM D4546-21, a soil is classified as highly expansive if its swell pressure is greater than 200 kPa.

It is important to note that these guidelines are general rules of thumb. Some expansive soils may not meet all of these criteria, while some non-expansive soils may meet some of them. Therefore, a comprehensive geotechnical investigation is essential to determine soil expansiveness. In addition to the physical and chemical properties mentioned above, various other factors can contribute to soil expansiveness, including:

- Climate: Expansive soils are more prevalent in arid and semi-arid climates, where there is significant moisture content variation.
- Topography: Expansive soils are commonly found in areas with flat or gently sloping topography, facilitating water pooling and soil infiltration.
- Vegetation: Expansive soils are less common in areas with dense vegetation cover, which helps regulate soil moisture content.

The aim of this paper is to portray the consequences of employing sustainable by-product materials readily accessible in Australia to stabilize expansive soils. These waste by-products, which are commonly found in Australia and can be utilized for the purpose of soil stabilisation, include: cement kiln dust (a by-product derived from cement manufacturing), blast furnace slag (originates from iron and steel manufacturing processes), quarry dust (a by-product of crushing stones in quarries), bagasse ash (obtained from burning sugarcane bagasse), rice husk ash (a residue of burned rice husks), fly ash (generated by coal combustion in power plants), and bottom ash (collected at the bottom of the furnace during coal combustion, which does not undergo burning). The production of the last two materials mentioned, namely fly ash and bottom ash, is expected to be limited in the near future due to the Australian government's decision to close most of the coal-fired power plants. Nevertheless, there will still be a significant quantity of fly ash and, notably, a substantial amount of bottom ash that will remain available for utilization in road construction for a couple of decades to come.

An extensive experimental program was conducted at the University of Technology Sydney using bagasse fibre, bagasse ash, bottom ash, fly ash, and eggshell powder in combination with hydrated lime or cement to improve the engineering characteristics of expansive soils. These laboratory tests were complemented by microscopic examinations, numerical analyses, and in-depth discussions. These pozzolanic materials contain significant levels of calcium carbonate, silica, and alumina. Multiple tests were carried out with these waste by-products to investigate their influence when combined with lime or cement, such as effect of curing time, wetting and drying cycles, particle sizes of bagasse ash, and the optimal blending ratios on treated soil properties. These tests included assessments of soil consistency, linear shrinkage, compaction factors, friction angle, cohesion, California bearing ratio (CBR), unconfined compressive strength (UCS), permeability, soil water characteristic curve (SWCC), tensile stress, Young's modulus, shear wave velocity, and microstructural properties. Rather than presenting an extensive set of results and lengthy discussions, this paper focuses on the key findings and typical results. Proper references are provided to access the original data and discoveries. To promote the use of waste materials available in Australia for mitigating the adverse effects of expansive soil, this paper also offers a list of recommendations.

2 WASTE MATERIALS IN AUSTRALIA

The National Waste Report, prepared by Pickin et al. (2022), clearly conveys that Australia produced an approximate total of 75.8 million metric tons of waste in the fiscal year 2020-21. This amount incorporates 25.2 million tonnes of building and demolition materials, 14.4 Mt of organic waste, 12.0 Mt of ash, 7.4 Mt of hazardous waste, primarily contaminated soil, 5.8 Mt of paper and cardboard, 5.7 Mt of metals, and 2.6 Mt of plastics. To put this in perspective, it averages out to 2.95 tonnes per person. As circular economy principles gain widespread acceptance among Australian state governments, the focus has shifted towards the practical application of these policies. This requires the ongoing practice of reusing and recycling materials, thereby diminishing the necessity for extracting new resources and minimising the volume of waste sent to landfills. Figure 1 displays the distribution of waste generation in Australia categorized by material type, presented as percentages.

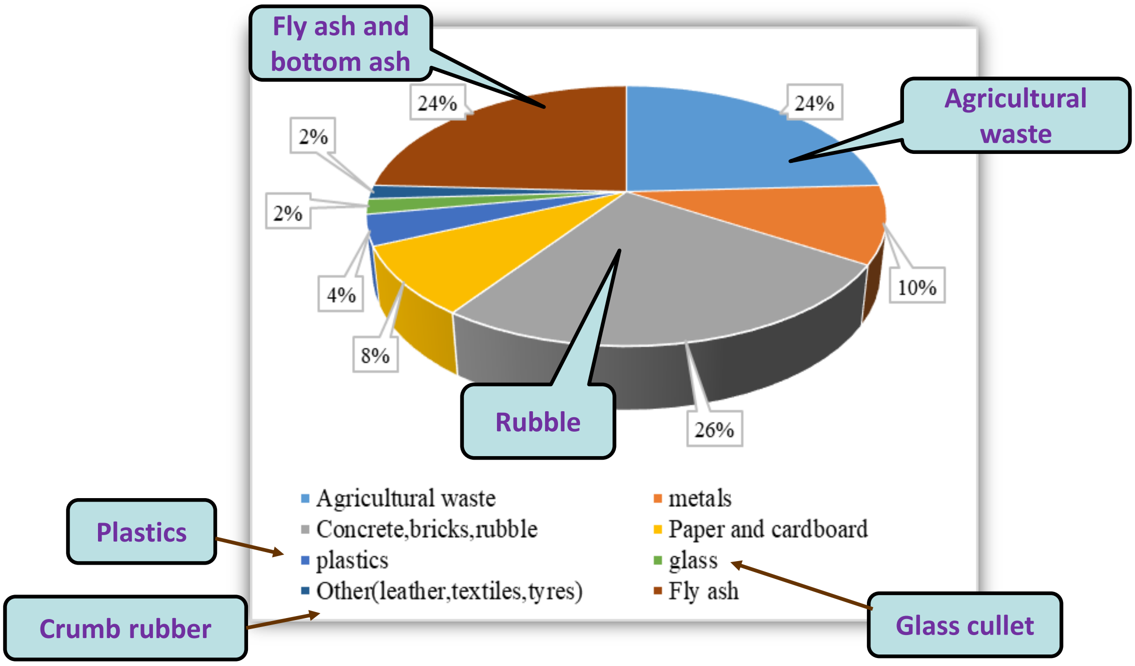


Figure 1: Generation of total waste in Australia by material category management in percentages (modified after Pickin et al. 2022)

3 USING DIFFERENT PROCEDURES FOR EXPANSIVE SOIL STABILISATION

3.1 ABUNDANTLY AVAILABLE BY-PRODUCT MATERIALS IN AUSTRALIA

In Australia, several common agricultural and industrial waste materials can readily be used to reduce the usage of lime and cement for soil treatment. Many of them are rich in silica and alumina with pozzolanic or cementitious characteristics. Some common waste materials that can be used in Australia for modification of weak soils are listed below:

- ✓ Sugarcane wastes (bagasse ash, bagasse fibre)
- ✓ Rice husk ash
- ✓ Coal fly ash, bottom ash, run ash
- ✓ Blast furnace slag
- ✓ Crushed waste glass
- ✓ Recycled tyre rubber, crumb rubber,
- ✓ Plastics
- ✓ Recycled construction and demolition waste
- ✓ Eggshell powder, eggshell ash
- ✓ Certain organic materials, such as palm oil ash, rice straw or coconut coir
- ✓ Cement kiln dust and quarry dust

It is important to note that the effectiveness of these waste materials may vary depending on the specific properties of the expansive soil and the intended application. Prior to using any of these materials for soil stabilisation, a detailed geotechnical analysis and laboratory testing should be conducted to determine the appropriate mix proportions and treatment methods.

3.2 SOIL STABILISATION APPROACHES

There are many methods available to curtail expansive soil swelling and shrinkage. There approached are summarised in Figure 2. Chemical stabilisation of expansive soils, particularly lime stabilisation in combination with waste materials, which adopted in the study, primarily involves the process of immobilising clay particles by cementing them in a manner that prevents them from expanding in volume.

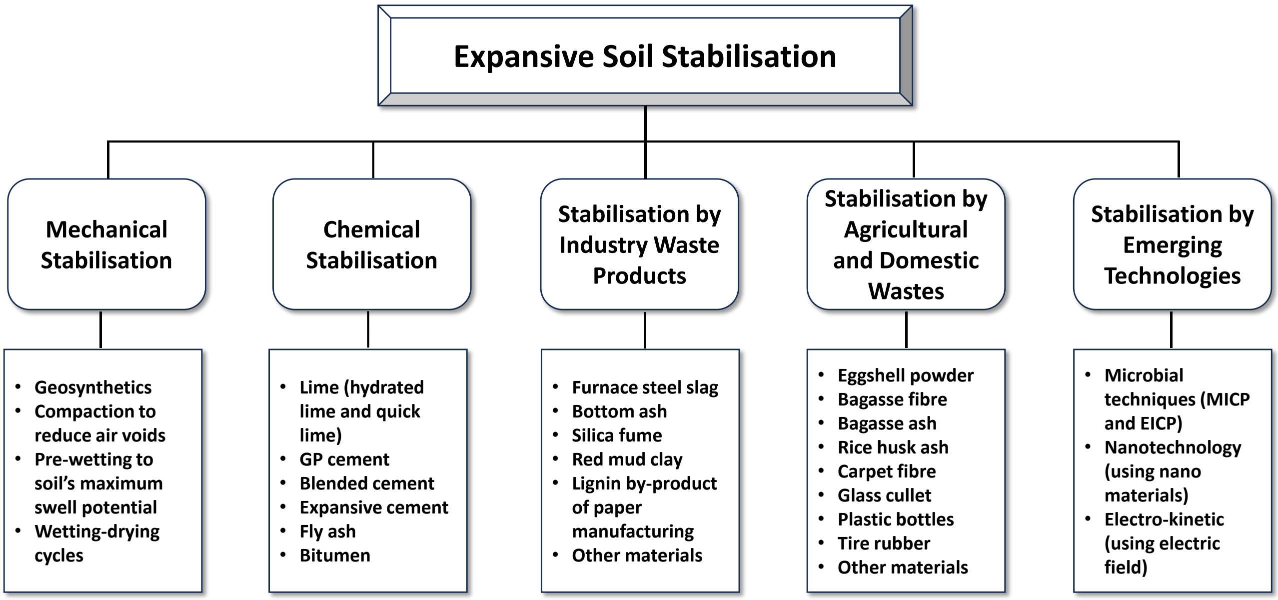


Figure 2: Various methods to improve expansive soil

Lime-soil stabilisation is a process of improving the properties of soil by adding lime. Lime reacts with the clay and silt particles in the soil to form cementitious bonds, which makes the soil stronger and more stable. Lime-treated soil was studied extensively in the literature (Firoozi et al., 2017; Little and Nair, 2009; Texas DoT, 2008). Many field and laboratory studies were conducted to evaluate the improvement of geotechnical properties of soil by addition of lime. The mechanism of treatment comprised hydration, cation exchange, flocculation agglomeration of soil particles and pozzolanic reaction to form calcium silicate hydrate (C-S-H) and calcium aluminate hydrate (C-A-H). The key factors affecting lime treated soil are type of lime, lime content, curing time, curing temperature and soil mineralogy. Soil-lime mixtures have advantages and disadvantages. Its advantages comprise significantly increase soil strength, reduce plasticity (increase workability) and increases soil durability. In addition, a considerable reduction in consolidation settlement and improve compressibility characteristics were observed.

However, there are some potential long-term problems associated with lime soil stabilisation. Leachate of lime over time, raising pH level of soil impacting on plant growth and soil microorganisms, sulfate attack when lime reacts with sulfates in the soil and forming ettringite, cracking potential as limed soil is more brittle, and environment impact such as reduced soil fertility and generating a lot of dust are a number of the disadvantages of lime-treated soil.

Several research studies have been carried out to offer recommendations for mitigating the negative impacts of these challenges. For example, strategies such as managing moisture levels, incorporating additives like fly ash or slag, or exploring alternative binding agents have been explored. Magnesium oxide and hydroxide can be considered as viable alternatives to lime due to their chemical properties, which make them promising candidates for addressing the drawbacks. Furthermore, some studies have shown substantial enhancements in soil strength, workability, and durability when using magnesium-based additives for soil stabilisation. Therefore, it is worthwhile to investigate the effectiveness of these materials in soil stabilisation through further research.

4 MATERIALS AND METHODS

The summary of materials and methods used by research team at the University of Technology Sydny (UTS) is given in Figure 3. The detailed material properties can be found in previously published papers and thesis by the researchers, conducted comprehensive investigations at UTS (e.g., Liet and Khabbaz, 2018, 2019; Liet et al., 2019; Le et al., 2023; Hasan et al., 2018; Alqaisi et al., 2020; Le, 2021).

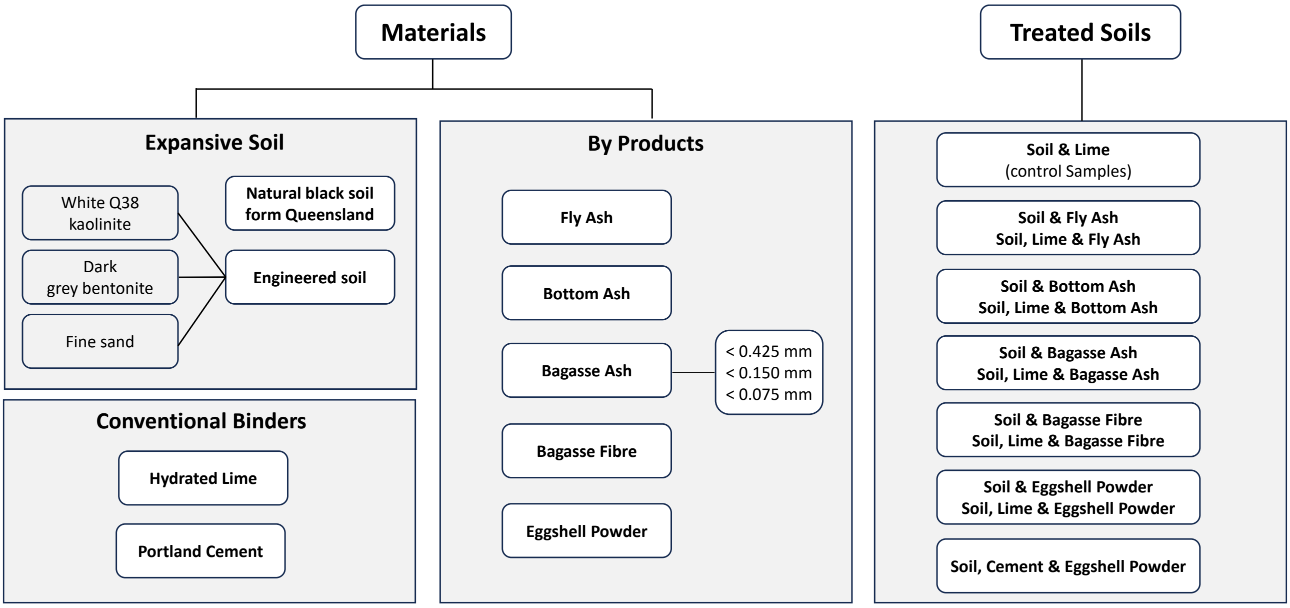


Figure 3: A simplified diagram representing the materials employed in the study

The main conventional binder, used for treatment of expansive soil has been hydrated lime. According to Adelaide Brighton Cement Ltd, the loss of ignition (LoI) of hydrated lime is around 24%. The specific gravity of hydrated lime is between 2.2 and 2.3. The pH level of hydrated lime is approximately 12, indicating its high alkalinity. Table 2 summarises the compositions of hydrated lime.

Table 2: Composition of hydrated lime

Composition	Chemical Formula	Value
Calcium Oxide	Cao	< 72%
Magnesium Oxide	MgO	< 1
Silicon Dioxide	SiO_2	< 2%
Aluminium Oxide	Al_2O_3	< 0.5%
Ferric Oxide	Fe_2O_3	< 0.6%
Calcium sulfate	$CaSO_4$	< 0.2

Incorporating hydrated lime into the soil results in an elevation of both soil pH and cation exchange capacity. The pH enhancement in clay supports time-dependent and temperature-related pozzolanic reactions.

In the experimental program at UTS, two different soil types have been used, including natural black soil, taken from Queensland (Figure 4), and engineered expansive soil. The properties of these soils are presented in Tables 3 and 4, respectively. The manufactured expansive soil examined in this research comprises three constituents: bentonite Active Bond 35, Kaolinite Q38, and Sydney fine sand. This soil is classified as a synthesized material, offering the advantage of precise control over the proportions of each component in every sample. Consequently, the soil samples are consistent and possess identical compositions, simplifying the process of comparing results.

Table 3: Properties of natural black soil taken from a road construction site in Queensland

Soil Characteristics	Average Value
Gravel (%)	< 0.1
Sand (%)	18.30
Silt/Clay (%)	81.65
Natural water content (%)	30.8
Liquid limit (%)	86
Plastic limit (%)	37
Plasticity index (%)	49
Linear Shrinkage (%)	21.7
Specific gravity	2.62-2.65
USCS classification	CH

To minimise the variations in soil properties and the presence of impurities typically found in in-situ soil, an engineered artificial soil was created for the experimental testing program. This artificial soil comprises bentonite (30%), kaolinite (65%), and Sydney fine sand (5%). The manufactured soil characteristics are detailed in Table 4.

Table 4: Properties of the engineered expansive soil

Soil Characteristics	Average Value
Plastic Limit (%)	30.9
Liquid Limit (%)	155
Plasticity Index (%)	124
Linear Shrinkage (%)	21.2
USCS Classification	CH
Maximum Dry Density (t/m^3)	1.34
Optimum Moisture Content (%)	28.8

Figure 4: Air dried eexpansive black soil in the soil laboratory at UTS

The study utilized bagasse ash with the following physical characteristics: a pH of 8.64, and a specific gravity of 2.32. The percentage of silica was 78.3% (Dang et al., 2016). The chemical compositions of bagasse ash collected from Australian Sugar Milling Council (ASMC) are summarised in Table 5.

Table 5: Chemical compositions of bagasse ash (after Dang et al., 2016)

Components	Content (%)
MgO	1.98
Al_2O_3	5.95
SiO_2	78.30
CaO	2.43
FeO	5.25
SO_3	0.89
K_2O	3.27
Na_2O	0.54
TiO_2	0.36
P_2O_5	1.03

The pH measurements indicated that about 5% hydrated lime content was sufficient for the short-term reactions (i.e., cation exchanges between soil particles and lime) to take place, defined as the optimum lime content to stabilise expansive soils as shown in Figure 5. while 18% (Bagasse Ash and %5 Hydrated Lime) combination was experimentally determined as the optimum additive combination for expansive soil stabilisation. Inspection of the UCS results of treated soils with curing time reveals that the strength development of soils mixed with lime and bagasse ash continued smoothly increasing after a long time of curing, meanwhile the compressive strength remained almost constant for lime treated soils as the curing time increased beyond 28 days. It is important to note that the low quantity of employed stabilisers has an insignificant effect on the strength gain of stabilised soils with time.

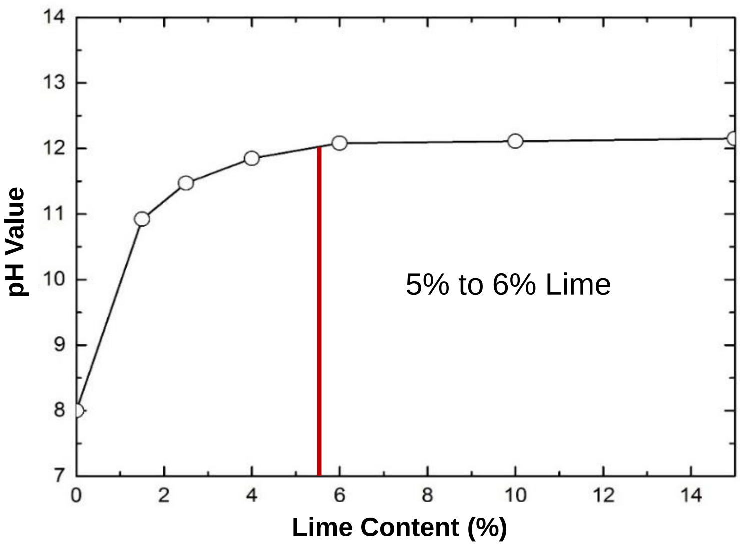


Figure 5: Variation in pH values when different amounts of lime are introduced into expansive soil after 1 hour of mixing (modified after Dang and Khabbaz, 2018)

The experimental program of the study can be categorized into several groups, as illustrated in Figure 6. An array of pH and electrical conductivity tests were conducted. Physical tests included the specific gravity, the particle size distribution, Atterberg limits and compaction tests. Meanwhile, mechanical tests in this study comprised linear shrinkage, free-swelling, consolidation, unconfined compressive strength (UCS), indirect shear strength (IDS), California bearing ratio (CBR), consolidation undrained (CU) shear triaxial, bender element, and suction measurements using filter paper method. Finally, the micro-structural analysis on samples from previous tests, consisting of four experiments :X-ray Diffraction, microscopic imaging and Scanning Electron Microscopy (SEM) tests followed by Energy Dispersive X-ray spectroscopy (EDX) analysis.

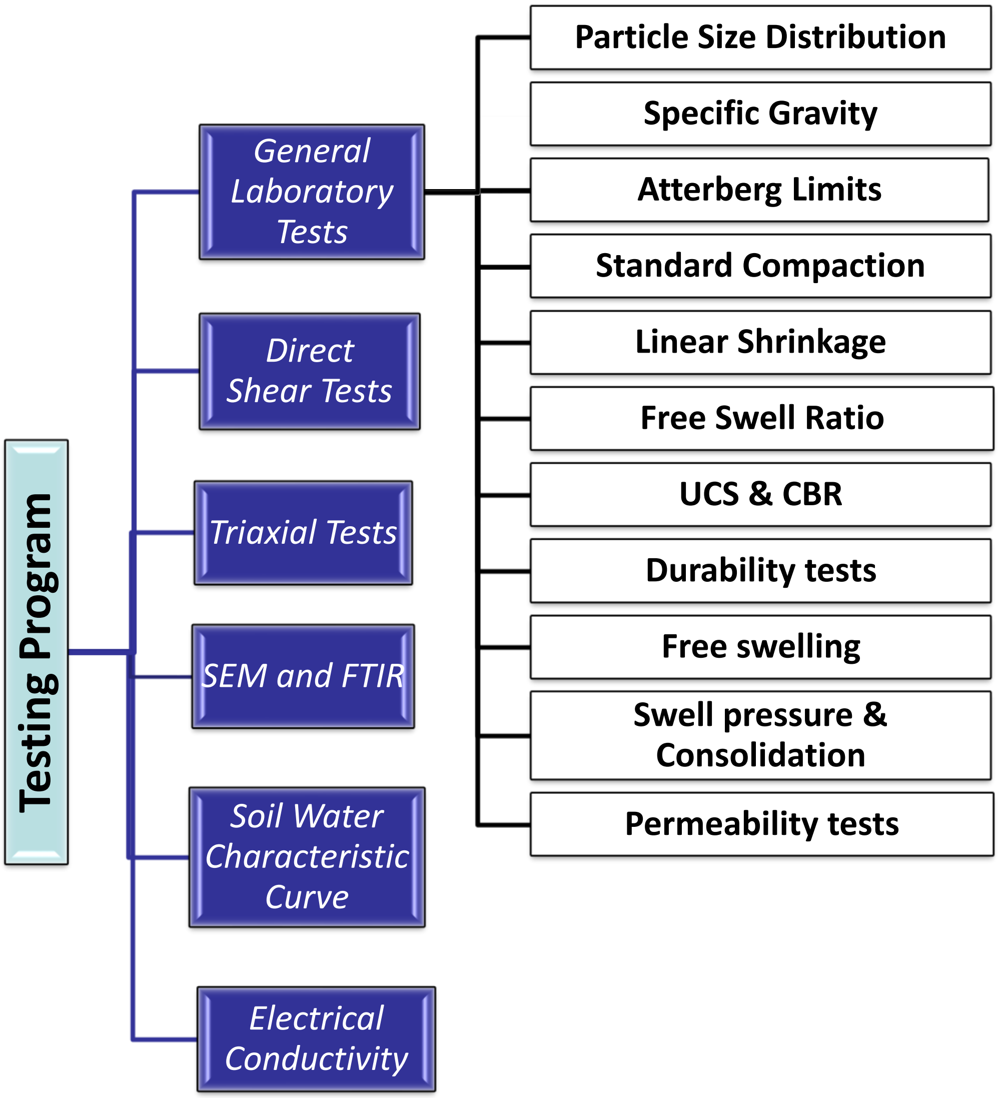


Figure 6: Summary of the testing program

5 RESULTS AND DISCUSSION

5.1 BAGASSE ASH AND HYDRATED LIME STABILISATION

An effective chemical stabilisation is a treatment method characterised by its convenient application, quick modification of soil characteristics, and enhancements in key soil properties, including moisture restraint, improved strength, and enhanced workability.

Bagasse ash is rich in silica and calcium oxide, both of which are recognized for their pozzolanic characteristics. When incorporated into expansive soil, bagasse ash undergoes a reaction with the soil, forming a cement-like substance that binds the soil particles together. This binding effect reduces the soil's capacity to expand and contract in response to fluctuations in moisture content. Research studies conducted by Le et al. (2019), Dang et al. (2019), and Hasan et al. (2018) have demonstrated that the inclusion of bagasse ash can enhance the strength and rigidity of expansive soils, diminish their plasticity and tendency to swell, and heighten their resistance to erosion. Moreover, bagasse ash stands as an environmentally friendly material, given that it is a by-product that would otherwise be discarded as waste. Its utilization in soil stabilisation can also curtail the demand for virgin materials, further lessening the environmental impact associated with construction projects.

Generally, the incorporation of hydrated lime and bagasse ash can alter the state of expansive soil from highly plasticity clay (CH) to elastic silt (MH) by reducing the liquid limit and increasing the plastic limit. Mixing hydrated lime and bagasse ash with expansive soil can lead to a decrease in both the maximum dry density and the optimum moisture content in the treated mixtures.

The linear shrinkage for black soil before treatment was 21.6%. The impact of adding bagasse ash and hydrated lime on the linear shrinkage of expansive soil at various curing times have been explored. As can be seen in Figure 7, the linear shrinkage of 13.5% bagasse ash (BA) plus 4.5% hydrated lime (L) was lower than employing 4.5 using hydrated lime only.

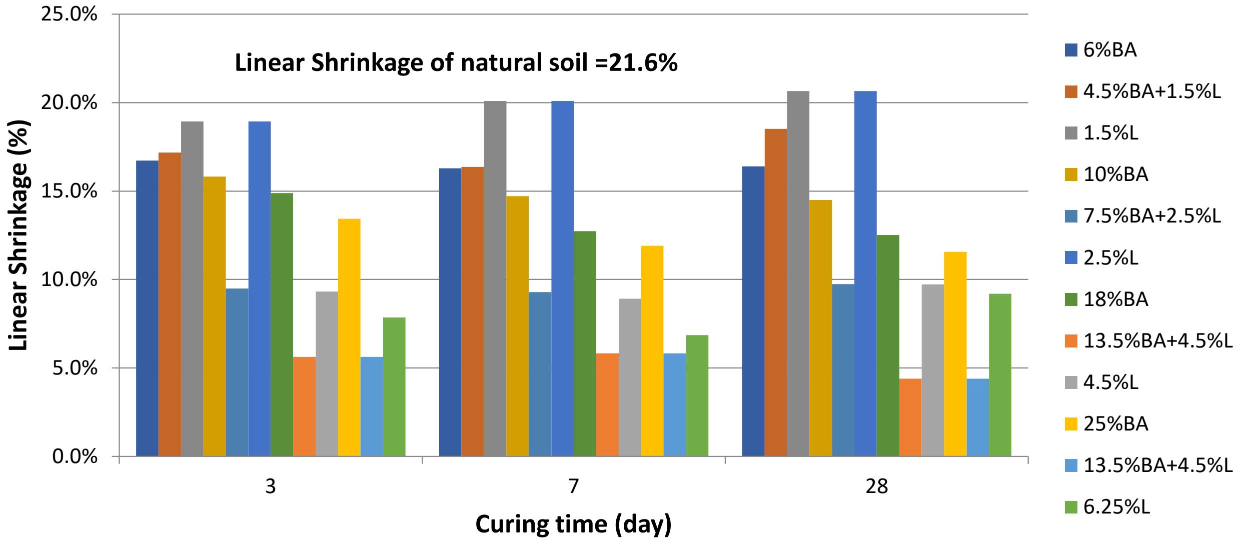


Figure 7: Effect of bagasse ash and lime on linear shrinkage of expansive soil at different curing time

The study revealed an effective lime content ranging from approximately 5% to 6%. One of the primary objectives of the experiments was to maximize the utilization of bagasse ash for lime stabilisation. As depicted in Figure 8, a ratio of 1 part lime to 3 parts bagasse ash proved to be optimal. A comprehensive series of tests was conducted to assess soil stabilisation outcomes, involving the removing bagasse ash particles larger than 425 microns before their introduction into the limed soil. The results of the unconfined compressive strength (UCS) tests are illustrated in Figure 9, clearly demonstrating that the addition of 15% bagasse ash significantly enhances the unconfined compressive strength of the samples.

The addition of hydrated lime stabilised expansive soil combined with bagasse ash resulted in a remarkable influence on the unconfined compressive strength and the compression curve and compression index of stabilised soil. The SEM results indicated the formation of cementitious products (i.e., CSH, CAH or CASH) as a result of the time-dependent pozzolanic reactions between clay or bagasse ash particles together with hydrated lime could be primarily responsible for the strength development and the stiffness improvement as well as the enhanced other geotechnical properties of treated soils.

The research team made a hypothesis that using finer bagasse ash can improve the strength of limed soil. Hence, in a study conducted by Le et al. (2023), the objective was to assess the use of bagasse ash with different particle sizes in combination with hydrated lime to stabilize expansive soils. The main findings from the tests conducted on bagasse ash in various maximum sizes of 75, 150 and 425 microns are as follows:

This study was part of an ongoing research program at UTS, focusing on mitigating the negative impact of expansive soils on pavements using sugarcane bagasse ash. The investigation involved testing for changes in shrinkage and strength properties of the soil mixture, including Atterberg limits, linear shrinkage (LS), and unconfined compressive strength (UCS) tests. The test results were correlated with three different maximum particle sizes of bagasse ash (i.e., 75, 150, and 425 microns).

The study found that the liquid limit of the mixture moderately decreased with a reduction in the maximum size of the bagasse ash particles, indicating that a reduction in the maximum particle size led to a decrease in the liquid limit. However, samples with larger bagasse ash particles (up to 425 microns) exhibited reduced linear shrinkage. When assessing the strength of lime-treated expansive soil with varying maximum bagasse ash particle sizes, it was observed that samples with a larger particle size (maximum 425 microns) outperformed mixtures with bagasse ash of smaller particle sizes. In other words, reducing the maximum particle size of bagasse ash below 425 microns did not result in an improvement in the unconfined compressive strength of ash-lime-treated soils.

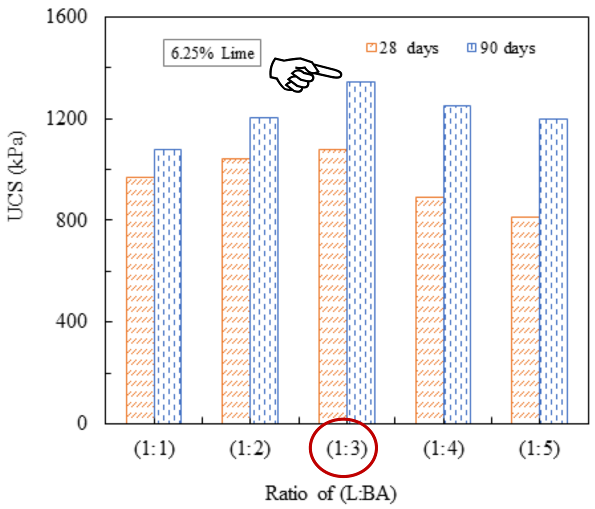


Figure 8: Unconfined compressive strength against lime (L) to bagasse ash (BA) ratio of Soil treated with 6.25% L and different BA contents ranging from 6.25% up to 31.25% (after Hasan et al., 2018)

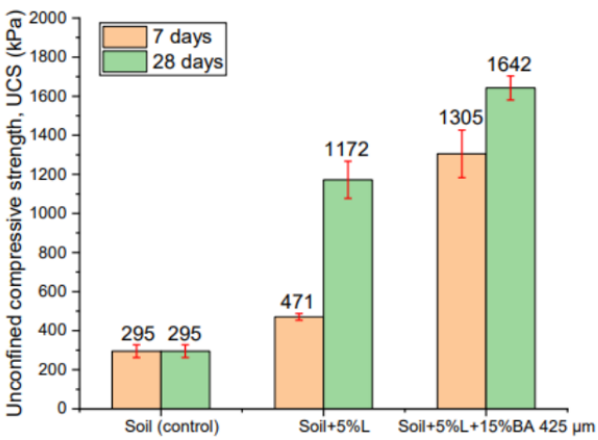


Figure 9: Unconfined compressive strength results for samples cured 7 and 28 days (after Dang and Khabbaz, 2018)

5.2 BAGASSE FIBRE AND HYDRATED LIME STABILISATION

A series of tests was conducted to investigate the engineering properties of expansive soil after being reinforced with randomly dispersed bagasse fibres and hydrated lime, in accordance with the testing methodologies and procedures detailed in Section 4 of this paper. Figure 10 illustrates the impact of incorporating bagasse fibres on the failure characteristics of lime-treated soils with varying fibre contents, ranging from 0% to 2%. It is evident that the inclusion of fibres enhances the ductility of the specimens. In this study, bagasse fibres with an average diameter of 3 mm were employed. Figure 11(a) displays a scanning electron microscopy (SEM) image of bagasse fibres, while Figure 11(b) provides a photograph of bagasse fibres used in the tests for being mixed with soil and lime at different volumes.

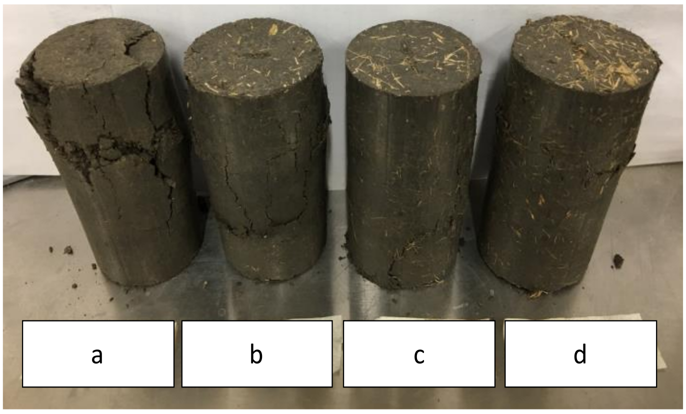
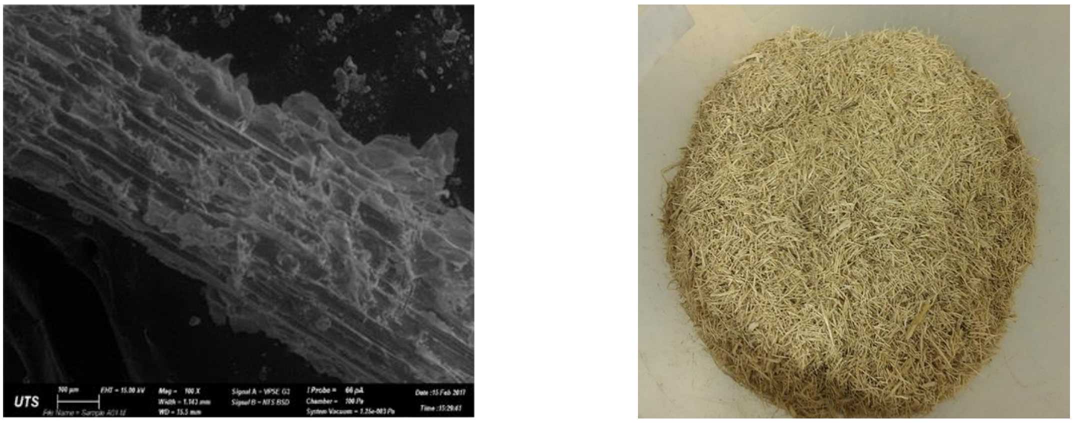


Figure 10: Effect of adding bagasse fibre (BF) on failure characteristics of 4% lime treated soils with: (a) 0% BF; (b) 0.5% BF; (a) 1% BF; (d) 2% BF



(a) Bagasse fibre (Dia: 0.3 mm to 3 mm) (b) Bagasse fibre (length: 0.3 mm to 13.8)

Figure 11: Bagasse fibre used in this investigation (a) SEM image, (b) photograph of bagasse fibre used in the test (modified after Dang and Khabbaz, 2019)

The following concluding statements have been excerpted from Liet and Khabbaz (2016), regarding the utilization of bagasse fibre in combination with lime for stabilisation of expansive soil samples.

- The results from standard compaction tests indicated a gradual decrease in the maximum dry density of soil samples as the content of bagasse fibre increased from 0% to 2%. When bagasse fibre was introduced in expansive soils, with or without lime treatment, there was a significant reduction in linear shrinkage as the additive content and curing time increased. The incorporation of bagasse fibre had a notable effect on the ductility of lime-treated soils, shifting the behavior of the stabilised soil from brittle to more ductile.
- When a higher lime content (i.e., 6% lime) was used for soil stabilisation, the addition of bagasse fibre to the limed soils enhanced tensile strength. However, the stiffness of the stabilised soil remained nearly unchanged when compared to soils treated with 6% lime without fibre reinforcement. It was observed that the California Bearing Ratio (CBR) showed a more noticeable increase in soil samples treated with the lime-bagasse fibre combination compared to samples reinforced with bagasse fibre or hydrated lime alone. The swelling behavior of expansive soil treated with a combination of bagasse fibre and lime was significantly improved, as lime stabilisation altered the physical and chemical properties of clay particles through cation exchange with lime. Furthermore, the compressibility properties of soils treated with lime initially decreased as the bagasse fibre content increased from 0% to 1%, but they increased when the bagasse fibre content exceeded 1%. The presence

of bagasse fibre reinforcement had a slight adverse effect on the water retention capacity of stabilised soils, though this effect was not significant. The improvement in air entry suction of stabilised soils was mainly attributed to changes in a small fraction of clay particles and enhanced pore size distribution resulting from lime stabilisation.

- As a result, the utilization of a combination of hydrated lime and bagasse fibre in expansive soil stabilisation not only improved the engineering properties of the soil but also reduced the environmental impact of agricultural waste by-products such as bagasse fibre. This combination also helped minimize construction costs by reducing the amount of lime needed.

5.3 BOTTOM ASH AND HYDRATED LIME STABILISATION

The UTS team conducted a comprehensive experimental study aimed at enhancing the geotechnical characteristics of limed soil and mitigating pressure on tailings dams by the use of bottom ash (Le, 2021). This readily available waste by-product, abundant in silica, was sourced from Eraring Power Station in New South Wales, Australia. Various proportions of bottom ash were mixed with hydrated lime to stabilize expansive soils. The research encompassed a wide range of experiments examining electrical conductivity, physical attributes, mechanical properties, and the microstructure of stabilised soils. It was followed by a numerical analysis of an embankment constructed on soft soils in Australia.

An innovative method for assessing electrical conductivity was employed to identify the optimal ash-to-lime ratio in the treatments, based on the pozzolanic reactivity of the ash mixtures. The test results revealed that when using 5% hydrated lime, the ideal proportion of bottom ash was 25%. However, when both bottom ash and bagasse ash were utilized, the respective ratios were 17.5% and 7.5%. When utilizing a bottom ash content exceeding 25%, it results in a reduction in the shear strength of the treated material as shown in Figure 12. This could be attributed to the surplus of bottom ash, which causes an imbalance between pozzolan and hydrated lime, consequently lowering the unconfined compressive strength (UCS) and California bearing ratio (CBR) after a 28-day curing period.

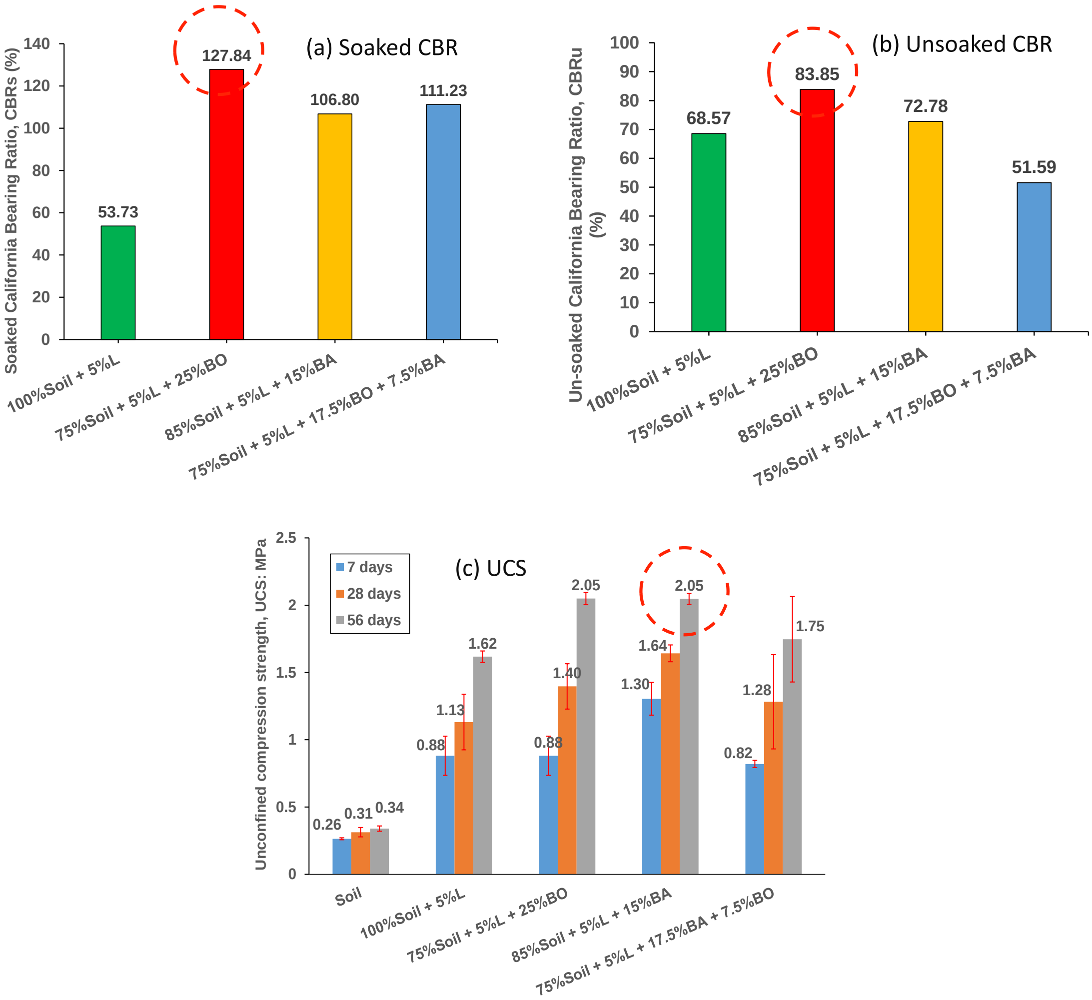


Figure 12: Characterization of treated soil using bottom ash, bagasse ash and hydrated lime

5.4 EGGSHELL POWDER AND HYDRATED LIME STABILISATION

Eggs, which are both nutritious and delicious, have become an essential part of the daily diet for many Australians. According to Safe Food (2022), to meet this demand, Australian egg producers generate approximately 17 million eggs on a daily basis, amounting to a total of 6.2 billion eggs annually. Given this the average Australian consumes around 240 eggs per person per year. Consequently, a substantial volume of eggshells is produced as waste material.

Eggshells are a readily available waste product generated by various sources, including hatcheries, fast-food establishments, poultry industries, egg product factories, and households. The accumulation of eggshell waste poses environmental concerns and often results in landfill sites. These eggshells can be transformed into eggshell powder (ESP) and serve as a soil stabilisation agent, as the primary component of eggshell powder is recognized to be calcium oxide.

A comprehensive experimental study at UTS revealed that incorporating eggshell powder (ESP) into clayey soil by itself had only a minor improvement on the geotechnical characteristics of expansive soils. However, when hydrated lime and ESP were combined, it had a substantial effect on the primary engineering properties of expansive soils. After conducting pH measurements, it was determined that around 5% of hydrated lime was the proper lime content for the stabilisation of expansive soils, facilitating cation exchanges between soil particles and lime. In addition, through experimentation, it was established that the optimal additive mixture for the stabilisation of expansive soil was 5% hydrated lime and 5% eggshell powder. More eggshell powder content did not increase the strength of limed soil.

In a separate investigation conducted at the University of Technology Sydney (UTS), eggshell powder and cement were utilized as stabilizing agents for expansive soil. This represents a promising methodology with the potential to enhance soil strength and mitigate linear shrinkage. Cement plays a pivotal role in expediting the pozzolanic reaction, thus leading to heightened strength. Augmenting the cement with 6% eggshell powder as a secondary additive to expansive soil resulted in a more pronounced enhancement of the stabilised soil's strength. Specifically, the Unconfined Compressive Strength (UCS) of soil specimens treated with both 6% eggshell powder and 6% cement exceeded that of samples treated solely with 6% cement by a noteworthy 25% as shown in Figure 13. Nevertheless, an examination of Figure 13 highlights the existence of a threshold for the quantity of eggshell powder that can be introduced, as an increment in the eggshell powder percentage initiated a reduction in the soil strength.

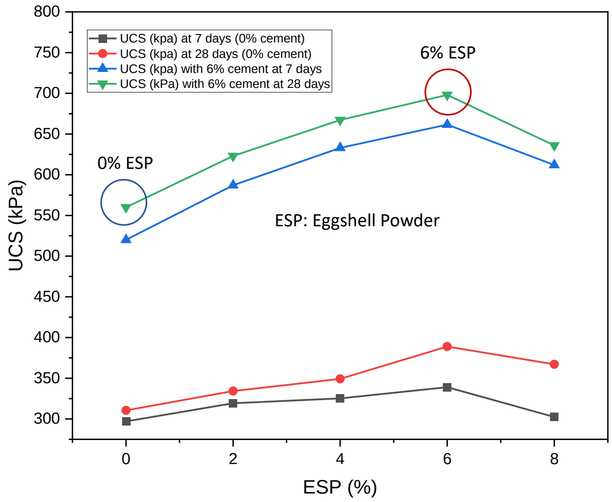


Figure 13: Effect of eggshell powder and cement on UCS of treated expansive soil at 7 and 28 days of curing

6 LESSONS LEARNT AND RECOMMENDATIONS

Practical recommendations for stabilizing expansive soil with lime, either with or without the addition of by-products, come with several lessons learned.

- One of the most effective strategies for mitigating the effects of expansive soils is to control the moisture content of the soil. This can be achieved through a range of techniques, such as the use of drainage systems to remove excess water from the soil, the use of moisture barriers to prevent moisture from reaching the soil, or the use of vegetation to absorb excess water.
- The application of lime should be executed with care to prevent the generation of dust and runoff.

- The best technique for stabilising expansive soil depends on the specific soil conditions and the desired outcome. For example, if the goal is to reduce the soil swell-shrink potential, lime stabilisation may be a better option. However, if the goal is to increase the strength of soil, cement stabilisation may be a better choice.
- Lime-Soil stabilised mix are useful to construct sub-base and base course for pavement. Lime treated soil is more suitable for warm regions where temperature is very high and for colder regions it is not suitable.
- Lime soil stabilisation is suitable for soils like clay, silty clay, clayey gravel and so on. However, it is not suitable for granular soils with some clay contents.
- A critical consideration when choosing the appropriate stabiliser is the amount of organic matter within the soil. Organic content can potentially interfere with the processes responsible for enhancing strength. If the organic content even exceeds 1%, supplementary additives may be required to counterbalance the cation exchange capacity of the organic material, as recommended by the Texas Department of Transportation (2008).
- Given the differences in physical and chemical interactions between the soil and the stabilizing agents, it is imperative to contemplate customized treatment strategies for the particular site. These strategies should be substantiated by evaluating soil-stabilizer combinations in simulated field conditions, as suggested by Little and Nair (2009).
- Although lime stabilisation is effective in improving the volume and strength characteristics of expansive soils, it is not without its constraints. According to Firoozi et al. (2017), these limitations are primarily associated with the presence of organic carbon and soluble sulfates. Research has shown that an excess of one percent organic carbon can disrupt pozzolanic reactions, leading to restricted strength enhancements. Conversely, the existence of sulfates raises more significant concerns because lime treatment in soils containing sulfate minerals such as gypsum ($CaSO_4$) and sodium sulfate (Na_2SO_4) can result in excessive swelling and pavement failures. These unfavorable reactions occur due to the formation of expansive minerals, notably ettringite, a mineral due to sulfate ions reaction with calcium and aluminum ions.
- Quicklime (CaO) is more effective than hydrated lime ($Ca(OH)_2$) for soil stabilisation; however, there is only a slight difference in the final results. If quick lime is applied, care should be taken otherwise dust and even burns may occur. Hence, in most of the cases hydrated lime is used either in dry powder form or by mixing water.
- Magnesium oxide soil stabilisation advantages: it can exhibit higher compressive strength, higher cementitious ability, and superior resistance to sulfate attacks than calcium-based systems. Generally, magnesium oxide (MgO) is more costly than calcium oxide (Cao), but it can exhibit higher compressive strength, higher cementitious ability, and superior resistance to sulfate attacks than calcium-based agents, which may reduce the maintenance cost and increase the durability of the stabilised soil.
- Cement can be utilized for soil stabilisation in a wide range of soil types, with the exception of soils containing more than 2% organic content or having a pH lower than 5.3, as specified in ACI 230.1R-90 (2009).
- Cement undergoes rapid hydration, leading to immediate strength development. Consequently, there is no requirement for a waiting period when using cement for stabilisation, and compaction of soil-cement samples is usually carried out within 2 hours of the initial mixing. However, cement-treated soils can become relatively brittle, leading to the potential for cracking and reduced flexibility. Lime-treated soils tend to exhibit greater flexibility and resilience.
- In order to prevent or reduce the risk of sulfate attack, it is advisable to conduct a sulfate content assessment on the soil prior to lime application. When high sulfate levels are detected in the soil, lime stabilisation may not be the most suitable option, or it may necessitate specific precautions. Potential measures to address this issue include reducing the lime dosage, incorporating a sulfate-resistant additive such as fly ash or slag, or considering alternative stabilisers such as cement or bitumen.
- Introducing 5% or more lime to expansive soil enhances its compressive strength and diminishes expansion by altering the structure of soil through processes such as cation exchange, soil particle flocculation, and the initiation of pozzolanic reactions (Le et al., 2018; Dang et al., 2016; Alqaisi et al., 2021).
- When sulfate is present in the soil, lime treatment can result in increased strength, but it can also lead to significant expansion due to the excessive formation of ettringite.
- Soils exhibiting sulfate levels exceeding 3,000 ppm, as suggested by Little and Nair (2009), should be regarded as potentially problematic. These soils necessitate special attention, starting from the choice of additives and extending through the entire process of mix design and construction.
- The stabilisation of acidic soil using lime, resulted in lower compressive strength than that of alkaline soil.
- There are significant challenges associated with managing treated expansive soils over the long term. Expansive soils can continue to undergo cycles of swelling and shrinkage for many years, and the properties of the soil can change over time due to weathering and other factors. As a result, it is important to monitor the behavior of expansive soils over the long term and to develop strategies for managing the soil behaviour as it changes over time.

- Only with careful assessment of the soil properties under traffic load in long term and the development of effective mitigation strategies, it is possible to design and build infrastructure that can withstand the challenges posed by expansive soils.
- Bagasse ash is generated as a combustion by-product from boiler of sugar factories comprises silica and can be employed as an admixture for treatment of expansice soil. It is considered pozzolanic and non-expansive material. Based on the results of pH measurements, it was found that about 5% hydrated lime was the optimum lime content to stabilise expansive soil, which required for cation exchanges between soil particles and lime to take place, meanwhile 18% hydrated lime-bagasse ash combination (1 lime to 3 bagasse ash) were experimentally determined as the optimum additive combination for expansive soil stabilisation.
- The unconfined compressive strength (UCS) of treated soils notably increased when additive content and curing time increased. The strength development of soils treated with bagasse ash and lime was higher than that of bagasse ash, or hydrated lime alone treated soils.
- The addition of hydrated lime stabilised expansive soil combined with bagasse ash resulted in a remarkable influence on the compression curve and compression index of stabilised soil.
- Bagasse ash can reduce the formation of ettringite in soil containing some sulfate. Bagasse ash contains silica, which can bind with the sulfate ions and prevent them from forming ettringite. Bagasse ash was more effective at reducing the swelling of soil caused by ettringite. Bagasse ash also contains aluminum ions, which can compete with calcium ions for the sulfate ions. This further reduces the amount of ettringite that can form.
- Bagasse ash can also improve the drainage of soil, which can assist to reduce the amount of water available for ettringite formation. the usage of sugarcane bagasse ash in treated soil assists in increasing the resistivity towards sulfate attack.
- Additives are the materials which are added to improve lime-soil mix to improve its strength. Some additives generally used are fly ash, slag, bagasse ash, bagasse fibre, rice husk ash, etc.
- Proper curing of the lime-soil blend is necessary to avert lime leaching into water sources. The lime-soil layer must harmonize with the surrounding vegetation and wildlife, considering its elevated pH levels.
- Inclusion of bagasse fibres in soil increases the strain at failure, and therefore makes the reinforced soil matrix more ductile.
- Class F fly ash is abundantly available in Australia, but it requires the inclusion of an activator such as lime or cement to create pozzolanic stabilised mixtures since it lacks self-cementing properties. When combined with lime, fly ash can be effectively employed for soil stabilisation purposes. According to Fatahi and Khabbaz (2013), the ratio of 1 part lime to 3 parts fly ash can effectively be used in soil stabilisation.

7 CONCLUSIONS

The performance of expansive soil after treatment with various eco-friendly waste materials, such as bagasse ash, bottom ash, and eggshell powder, both with and without conventional binders like lime and cement, has been examined. These materials offer potential for sustainable infrastructure development, reducing the need for traditional stabilizers in expansive soil treatment. The findings suggest that when combined with hydrated lime, these by-products not only improve the geotechnical properties of expansive soils but also address the impending environmental challenges associated with waste material disposal. Through research and laboratory studies, it has become evident that sustainable by-product materials show significant promise in terms of increased longevity, cost-efficiency, and long-lasting environmental benefits compared to conventional lime or cement-based agents for addressing expansive soil issues. These materials enhance soil strength, reduce swelling potential, improve soil ductility, and control deformation over time. However, the implementation of these sustainable materials in practice is not yet widespread among construction companies and road authorities in Australia.

Lime stabilisation for expansive soil, whether used alone or in combination with marginal materials, may not be the most effective approach in regions where wetting and drying cycles have a significant impact. Another concern arises from the presence of sulfates in the soil. When lime reacts with these existing sulfates in the soil or groundwater, it can lead to the formation of ettringite. Sulfate attack can result in soil expansion, cracking, and structural deterioration, thereby diminishing the soil's strength and durability. The influence of cyclic loading needs to be carefully considered, especially when addressing shrinkage effects on roads using lime. The use of ashes, fibres, and other waste materials helps alleviate the pressure on landfills. However, a thorough investigation into the long-term performance and durability of stabilisation is needed. The practical application of by-products as road construction materials should be assessed through field tests. Collaboration with road authorities is undoubtedly essential for effectively incorporating agricultural and industrial waste materials into road foundations.

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GROUND IMPROVEMENT SUSTAINABLE BY NATURE

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ABSTRACT

The construction industry faces significant challenges today due to its high energy consumption and the resulting elevated carbon dioxide emissions. Efforts are underway through global initiatives and evolving frameworks to establish comprehensive environmental and sustainability policies, aiming to guide the construction sector towards a more sustainable development pathway. Geotechnical engineering within the realm of infrastructure is expected to play a critical role in ensuring the safety and stability of superstructures within a sustainable framework, both in the present and the foreseeable future. This article seeks to explore the potential for sustainability in infrastructures through ground improvement techniques. It sheds light on ongoing global research and development efforts aimed at promoting sustainability. The article briefly introduces a case study involving the rigid inclusions type Controlled Modulus Columns (CMC), emphasizing ongoing research and development focused on the reutilization of CMCs under wind turbine foundations during the repowering phase. The repowering phase becomes imperative when onshore wind turbine foundations require replacement after reaching their typical operational lifetime of 25 years, as stipulated by international standards and guidelines.

1 INTRODUCTION

The construction industry, marked by substantial energy consumption and significant greenhouse gas emissions, stands categorized as unsustainable. Global data underscore the criticality of this challenge, with the construction sector responsible for 36% of global energy consumption and 39% of energy-related carbon dioxide (CO_2) emissions (UN Report, 2017). In Australia, the construction sector has a significant impact on the landscape, as evidenced by arround 9% share of GDP in February 2021 (Australian Bureau of Statistics, 2022), making it the second- largest industry after the mining sector. Notably, Australia, representing merely 0.33% of the world's population, ranks among the highest per capita emitters of greenhouse gases globally (Man et al., 2017). The construction markets in Australia exhibit a direct relationship with population growth. Projections by (Australian Bureau of Statistics, 2023) highlight a significant surge in Australia's population, potentially reaching approximately 49.2 million by 2066 almost double the current populace. This notable demographic growth is anticipated to propel demand in the economy, especially within the construction sector, with a specific emphasis on housing. Concurrently, Australian policymakers are proactively enacting stringent measures to curb carbon emissions and advance sustainability across diverse projects (Talberg et al., 2016).

The definition of sustainability within the construction sector varies significantly, leading to various interpretations and applications. The origins of "sustainable construction" trace back to the inaugural international conference in the United States in 1994, primarily advocating for sustainability in construction to conserve natural resources and mitigate global warming. Over time, sustainability has evolved, becoming a pivotal aspect in research and across organizational levels, from individual companies to entire nations. Diverse dimensions of sustainability have been integrated into the construction market, encompassing responsible practices aligning economic growth, social equity, and environmental stewardship. These practices include waste reduction, utilization of eco-friendly materials, energy efficiency, optimal design and construction processes, and best management practices. The overarching recognition of sustainability as a crucial objective, particularly emphasizing environmental approaches, is underscored by the majority of nations signing the Paris Agreement (UNFCCC, 2015). A comprehensive review of various sustainability models and measurement approaches is extensively documented in a thorough literature review (Yu et al., 2018).

Ground improvement techniques, known for their inherent material efficiency compared to traditional foundations, are often regarded as sustainable methods. This paper aims to contextualize ground improvement methods within the realm of sustainability, delving into ongoing research concerning diverse sustainability aspects related to the reuse of CMC during the repowering of onshore wind turbines. Ground improvements are usually referred to as sustainable methods, as they are naturally having less materials compared to others classical foundations. This paper attempts to present the ground improvement techniques in sustainability frame and present part of research done on different aspects of sustainability in geotechnical engineering with particular emphasis on foundation engineering and ground improvement.

2 SIMPLIFIED SUSTAINABILITY APPROACH IN CONSTRUCTION

The term "sustainability" has gained considerable importance in recent decades, attracting attention in both scientific discourse and practical application. In its broadest sense, it stands for the principle of development that meets present needs without compromising the ability of future generations to meet their own needs (Brundtland Report, 1987). The construction industry's interpretation of sustainable development has evolved into a comprehensive framework that encompasses environmental, social, and economic dimensions. Initiatives that aim to minimize harmful environmental impacts through strategic waste reduction, reuse, and efficient management not only benefit the environment but also have direct positive societal impacts (Bamgbade et al., 2017). In the construction sector, achieving sustainability requires the integration of economic dimensions to meet the principles of the circular economy and to resonate with decision makers. Therefore, an integrated three-dimensional approach that incorporates social, environmental, and economic considerations into project implementation and gives equal importance to all aspects of sustainability is imperative.

At the project-specific level, tailoring the sustainable framework to specific requirements and applicability is critical. Assessing and evaluating the level of sustainability in construction is a multi-faceted challenge. There are various approaches and analysis methods to identify and improve sustainability indicators and their impact on construction projects. Sustainability indicators, a recurring term in any sustainable framework, represent measurable parameters or metrics used to evaluate the tripartite approach to construction activities throughout their life cycle. These indicators provide insight into the sustainability performance of a construction project and help stakeholders make informed decisions to mitigate negative environmental, social, and economic impacts.

In the infrastructure industry, accounting for uncertainties in ground behaviour and mitigating geohazards in the context of sustainability requires a delicate balance in the choice of sustainable model. The chosen model must also be reliable, resilient, and adaptable to technical challenges (Basu et al., 2015). Key indicators for assessing infrastructure sustainability were identified from the relevant literature (Shen et al., 2011). These indicators include environmental, social, and economic factors and highlight aspects such as water quality impacts, public safety, market supply and demand analysis, waste management, local development impacts, financial risk, carbon footprint, employment opportunities, project budget, energy savings, public sanitation, and technical benefits.

Acknowledging the inherent challenges at various scales within each project, often constrained by tight design schedules and stringent safety standards influencing design and construction approaches, we present these simplified key considerations. These serve as a global reminder of the essential steps for effective sustainability integration encompassing all stakeholders in a construction project leaders, management, engineers, and construction teams. Each of the indicators in (Figure 1) could be measured effectively by all construction teams.

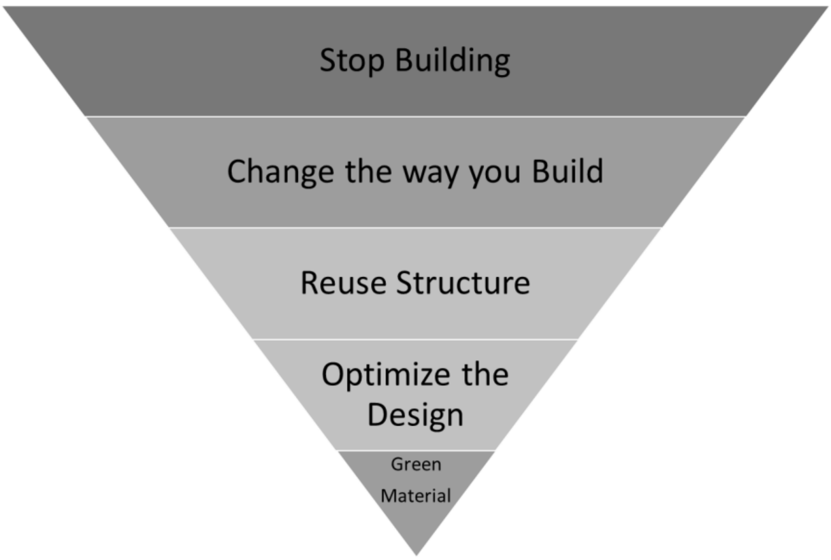

Figure 1: Simplified sustainability approach

Stopping construction abruptly is a drastic measure to curtail material waste, energy consumption, and pollution. Nonetheless, it is not widely viewed as a feasible approach for advancing sustainability in the construction sector. A complete freeze on construction is not practical, especially given projected population growth and significant infrastructure and housing needs. Moreover, this approach would adversely impact other important sustainability dimensions such as the economy and society by hindering economic growth, urban development, employment prospects, and societal benefits.

In the subsequent section of this article, the authors elaborate on the indicators illustrated in Figure 1.

3 CHANGE THE WAY YOU BUILD IN INFRASTRUCTURE

Infrastructure development is a cornerstone of modern society. It provides the necessary framework for economic growth, urbanization, and social well-being. Traditional methods for building infrastructure, including deep foundations, shallow foundations, and pile foundations, have been the traditional approach for decades. However, the increasing demand for sustainable and cost-effective solutions has led to the use of ground improvement techniques becoming more popular. Ground improvement offers alternatives that optimize foundation performance, reduce CO2 emissions, decrease resource consumption, and improve long-term environmental sustainability.

Ground improvement methods have evolved considerably over the past five decades. They are now recognized as a major sub-discipline of geotechnical engineering (Schaefer et al., 2012). Ground improvement is used primarily because of the increasing need to utilize marginal sites for new construction and to mitigate the risk of failure or potential poor performance. The increase in applications of ground improvements is also related to their economic and environmental benefits. Some of these techniques do not involve mixing or drilling materials into the soil; rather, the mechanical action of the equipment improves the mechanical properties of the soil. Converting large areas of difficult soil conditions into buildable areas is one of the incomparable advantages of soil improvement over traditional geotechnical solutions. There are numerous solutions for soil improvement.

Many ground improvement techniques not only offer compelling economic advantages but are also beneficial from an environmental perspective. These methods result in significant material savings, waste reduction, less environmental disturbance, shortened project duration, all of which are conducive to sustainable land development. The following table describes the environmental impacts of each land improvement method.

Table 1: Sustainability in Nature of Ground Improvement Technics

Category	Ground Improvement Method	Sustainability Indicator
Densification	Dynamic Compaction Vibro Compaction Surface Compaction	Rearrangement of soil particles, enhancing mechanical characteristics for future superstructure development No material injection, no waste production, environmentally friendly Economical savings through material reduction and ground stability
Consolidation	Preloading without drains Preloading with vertical drains Vacuum consolidation	Natural soil matrix enhancement for future superstructure development No material injection, no waste production Biodegradable drains Economical savings through material reduction and ground stability
Soil Mixing	Mass Soil Mixing Deep Soil Mixing Jet Grouting	In-situ soil mixing for improved mechanical characteristics in superstructure development Reuse and recycling of on-site materials, minimizing waste and environmental contamination Energy conservation and cost efficiency through reduced material and labour requirements
Reinforcement	Rigid Inclusions Stone Columns / Dynamic Pillars Bi-Modulus Columns	Introducing rigid or semi-rigid elements in structure matrix to transfer load efficiently and reduce stress in the ground Lower carbon dioxide emissions, faster construction, minimal construction materials, no need for steel connections with foundations

4 OPTIMIZING THE DESIGN

Design optimization is a process that involves improving various aspects such as efficiency, durability, cost effectiveness, and environmental impact, while considering sustainable goals. Integrating sustainable principles into the design phase is paramount to achieving the overall sustainable development goals. However, there is no singular rule or predefined standard for design optimization in this domain. Different efforts could lead to optimization, such as:

- Parametric and sensitivity studies involve a systematic examination of design parameters and their impact on project outcomes. This approach requires time and appropriate tools to allow designers to perform multiple iterations during the design phase. In this way, designers can refine the design, reduce unnecessary dimensioning, and optimize critical aspects of the project.
- The frequency of geotechnical testing is increased. A thorough understanding of soil conditions, properties, and behaviour under different circumstances during the design phase allows engineers to anticipate challenges and make informed design decisions more accurately. By minimizing uncertainties, engineers can optimize the design, select appropriate materials, and design foundations and structures that are both safe and efficient.
- Building Information Modelling or the new trending digital information system. Powerful for creating a digital representation of the project that integrates various aspects such as geometry, spatial relationships, geographic information and quantity data. Using BIM, designers can simulate the project, identify potential clashes or conflicts, optimize the layout for efficiency, and evaluate the project's impact on the environment. BIM enables multidisciplinary collaboration, which is essential for holistic design optimization. By using BIM, designers can comprehensively visualize the project, make informed decisions, and iteratively refine the design, ultimately leading to a more sustainable and efficient outcome.
- Involving experienced engineers in the design process, as they can draw on their expertise to implement innovative design approaches, alternative materials and construction methods that align with sustainability goals to ensure a more efficient and sustainable project outcome.

Choice of Foundations

The choice of an appropriate foundation type stands as a critical decision within infrastructure industry. This selection process depends on a multitude of factors, such as soil conditions, bearing capacity, structure type, and the nature of applied loads. Geotechnical engineering conventionally employs four fundamental foundation types, expounded upon in (Figure 2). The decision regarding the foundation type is significantly influenced not only by technical considerations but also by traditional practices and prevailing knowledge within the construction sector.

The adoption of distinct foundation approaches can also be categorized as an optimization of the design. As an illustrative instance, embracing a ground improvement strategy utilizing Controlled Modulus Columns (CMC) represents a sustainable alternative to classical solutions. This is notably apparent when shifting from an initial design centred on pile foundations to the CMC approach or transitioning from a larger shallow foundation to a more compact area through the integration of CMC. The incorporation of CMC stands as an effective means to optimize the design proficiently. The typical foundation types used in the industry are listed in (Table 2).

Table 2: Foundation types and features

Foundation type	Concept
Shallow Foundation	Designed to transfer the load of a structure to the near-surface soil layer. This is a common type of foundation used in the construction of buildings and various structures. However, if the stability and settlement of the structure cannot be guaranteed, alternative foundation solutions need to be considered.
Deep Foundation	Engineered to transfer the entire loads of a structure to deeper soil or rock layers with better properties, especially when the surface soil conditions are inadequate to withstand the applied loads. Load transfer to these deeper strata is achieved using rigid elements like piles that are connected to the structure, providing the necessary stability and load-bearing capacity.
Mixed Foundation	Combined elements of both shallow and deep foundations to enhance the stability of structures and reduce additional settlements. However, this approach is more complex and costly due to the specialized equipment required and the extended construction time. Despite the advantages in stability improvement, these techniques often demand careful consideration of additional loads such as lateral and dynamic forces during the design phase.
Soil Reinforcement (CMC)	An intermediate solution between shallow and deep foundations, aiming to minimize settlement and enhance the bearing capacity of soil foundations for various superstructures. This technique resembles a mixed foundation but lacks a structural connection with the foundation above. Instead, it typically incorporates a load transfer platform. The significant advantage of this method lies in its construction simplicity, reducing column diameter in comparison to piles, and proving highly effective, particularly in challenging soil conditions.

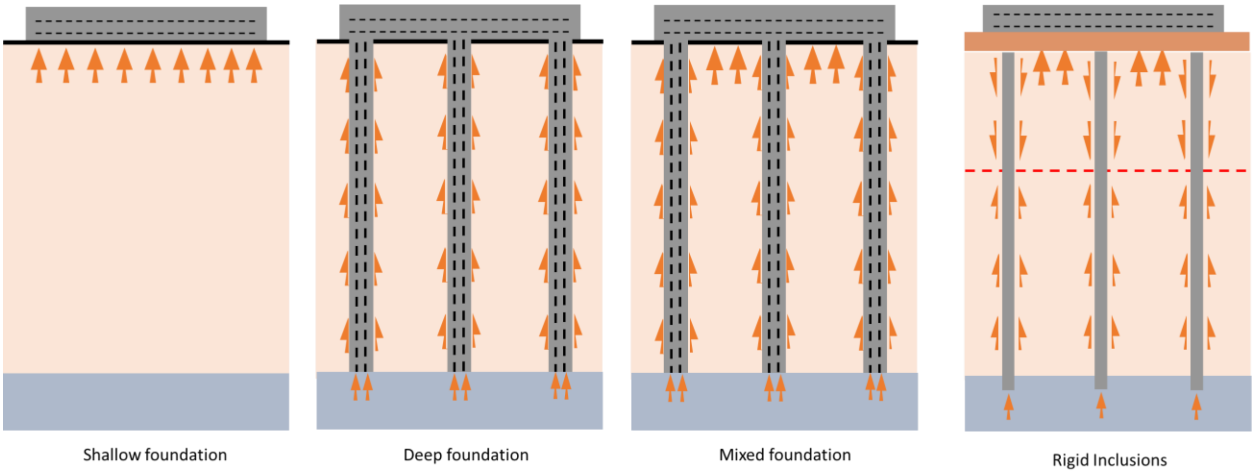

Figure 2: The various types of foundations (ASIRI, 2013)

5 REUSE THE STRUCTURE

The integration of reused building components or foundations into a construction project is a natural measure to ensure sustainability in the construction industry, which includes environmental, economic, and social aspects.

From an environmental perspective, this practice significantly reduces the need for new materials, which leads to the conservation of natural resources and the reduction of energy consumption in the production of new materials. It also effectively curbs construction-related waste.

Economically, reusing an entire structure or certain components proves to be a financially prudent approach when contrasted with the purchase of new materials. Not only does this approach result in significant cost savings, but it also reduces the cost of waste disposal, adding to the economic benefits. In addition, the burgeoning market for recycled materials improves the economic outlook and promotes sustainability.

On a social level, reuse of materials can stimulate local job creation by creating employment opportunities in salvage operations, materials processing, and renovation work. In addition, the preservation of architectural and historic elements as well as vital elements for energy production such as wind turbines.

Case Study

Onshore wind turbines have emerged as a significant player in the construction market, representing the fastest-growing renewable energy technology worldwide over the last four decades. They stand as a cornerstone of green energy sources on a global scale, and ensuring a steady supply and enhancing their productivity is crucial to reduce reliance on fossil fuels. Typically, onshore wind turbines have an expected lifespan of around 20 years, a duration that recent studies suggest might be slightly extended based on various factors, including the structural fatigue life. Given the pressing environmental concerns, it becomes imperative to sustain renewable power generation by replacing wind turbines that have reached the end of their operational tenure, a process known as re-equipping or repowering. These efforts are seen as essential, particularly in Europe, aiming to diminish dependence on fossil resources. In 2016, 12% of the existing wind turbines in Europe had been in operation for at least 15 years. This percentage is projected to escalate substantially by 2030 as illustrated in Figure 3 (a). Consequently, it is foreseeable that the repowering rate will surge to maintain current energy production levels and augment them in the future, as indicated by the 2030 scenarios for wind energy in Europe as shown inFigure 3 (b).

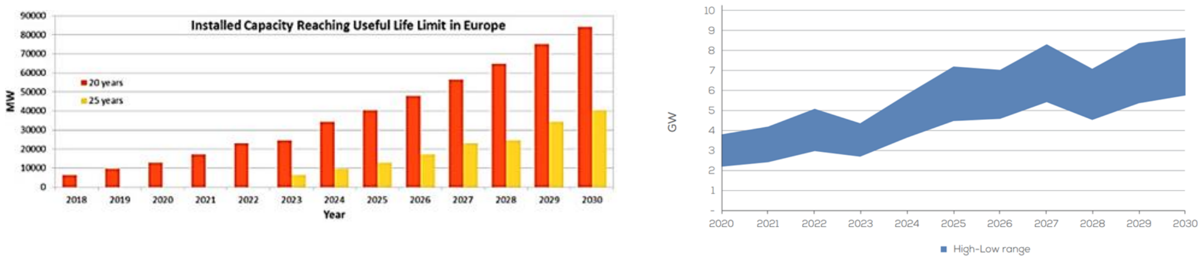

Figure 3: (a) Installed wind power capacity in Europe reaching the end of its useful life, (b) Repowering volumes in Europe to 2030

Repowering, in essence, involves the removal and replacement of wind turbines as they reach the end of their operational life. Various strategies have been devised to achieve this objective:

1) Full dismantling and removal of the wind turbine, followed by the construction of a new foundation at a different location.

2) Full dismantling and removal of the wind turbine, succeeded by the construction of a new foundation at the same location.

3) Reuse of the entire wind turbine foundation or a portion of it, facilitating the installation of a new-generation wind turbine while preserving the existing foundation structure.

Initiating the third strategy as a sustainable solution, FEDRE is a research project with an industrial background in which Menard actively participates. The project has two main focuses: First, it aims to find an innovative solution for the reuse of gravity foundations by replacing wind turbines at the end of their lifetime with new generations of onshore wind turbines that are relatively massive compared to the old generation. Second, it explores the possibility of optimising the design of current onshore wind turbine foundations to support multiple generations of wind turbines in the future as part of the proposed repowering solution will be explored. The project targets three markets: environmental, economic, and applied research.

Environment

Regarding the environmental aspect, repowering strategies will result in less concrete being used for foundations. This means that less cement will be produced, since one ton of cement needed to produce reinforced concrete requires the emission of approximately 807 kg of CO_2 (Chen et al., 2010). In addition, the production of green energy is maintained and increased, reducing fossil energy.

Economic

Regarding the economic market, a rough estimate of the construction work for an onshore wind turbine can be up to 10% of the total cost, i.e., 100 k€ per installed MW. The repowering of an existing foundation would allow a saving of about 1/4 to 1/3 of the construction cost, i.e., between 150 and 200 k€ for the change from a 3 MW to a 5 MW wind turbine.

For the French market, where the installed base is known, repowering could concern 1,200 to 1,300 wind turbines out of 3,200 wind turbines reaching an age of 20 years by the end of 2029. Thus, repowering could generate in 2029 in France a turnover of about 8 000 k€ and 50 jobs in the different partners of the FEDRE project (14 k€ per installed turbine MW).

By 2028/2030, repowering in Europe is expected to include a capacity of 6 000 MW. With a market share of 5%, which is realistic given the international activities of the various partners, the projected export turnover would be in the order of 4,200 k€.

By reducing this total cost and in the context of accelerating repowering projects, this will lead to a very important outcome for the economic balance of the projects. Moreover, it will serve as an advertisement for onshore wind energy in France and lead to more investments in renewable energies.

Research Field

Reusing part of wind turbine foundation strongly depends on the integrity and capacity of the existing foundations for the long-term success of repowering efforts. This therefore requires a thorough, detailed, and comprehensive review of existing foundations, including strength, serviceability, and fatigue analyses.

The project strategy is essentially divided into 6 lots, which denote different subject areas that represent the transition from research to industry to achieve the project objectives. The lots are supported by the expertise of the industrial partners and the research orientation of the project. This combination is aligned as follows:

- GEOMAS at INSA Lyon: a research laboratory in civil engineering and materials, with a geotechnical focus on rigid inclusion technique through the various research activities related to ASIRI & ASIRI+
- MENARD GROUP: worldwide specialist in ground improvement and soil reinforcement, active in Design & Built as well as selected research activities
- ANTEA GROUP: geotechnical consulting firm and laboratory
- NORDEX&ACCIONA: manufacturer and operator of wind farms
- CTE Wind: designer of wind turbine foundations
- PAREX: specialist in construction materials

The main lots of the project can be summarised as follows: (1) The demonstrator, (2) the physical modelling, (3) the numerical modelling of the mechanisms observed and highlighted in the first two cases, (4) the transfer from research to engineering, (5) the searching for solutions to improve the maintenance and reuse of foundations for repowering phases, (6) the valorisation of the project.

The research findings from this project have been meticulously recorded in various scholarly papers and related project activities, all of which are dedicated to wind turbine repowering. While this paper does not address specific research findings, it serves as a focused platform for discussing sustainability in the context of a simplified case. In the next section, we will present the potential repowering solution.

Repowering Solution

Diverse foundation types are utilized to provide support for onshore wind turbines, including gravity foundations, deep foundations, rigid inclusions, and other ground improvement techniques. In the European context, rigid inclusions find extensive use, especially among solutions necessitating deep foundations. Notably, in France, recent statistics reveal a substantial adoption of rigid inclusions in wind turbine foundations, accounting for up to 24% of the foundation market, in stark contrast to the mere 0.6% attributed to pile foundations. In FEDRE project, CMC supporting a concrete gravity foundation is the selected foundation type for this initiative.

The repowering strategy described in this project relies on several critical components: the redesigned wind turbine structure, the onshore wind turbine foundation, and the soil reinforced by CMC. The project involves reusing part of the existing foundation and increasing its dimensions in a mushroom-like configuration. Both the new and old foundations were extensively studied and validated before this approach was applied. The validation process included numerical analysis and careful physical modelling in a small laboratory (Modu, 2022). The foundation expansion will be performed on soil already loaded by the existing wind turbine and additionally by the enlarged area of the new foundation. Given the structural nature of CMC, the lack of a direct connection between the soil and the concrete foundation is a critical factor in the success of the project. Logistically, it becomes feasible to introduce new CMC to extend the coverage of the foundation, effectively expanding the reach and influence of the reinforced soil. This extension enables the foundation to be bolstered, precisely tailored to meet the heightened load demands associated with wind turbine repowering. The proposed approach for the repowering phase in the project is outlined is shown in Figure 4.

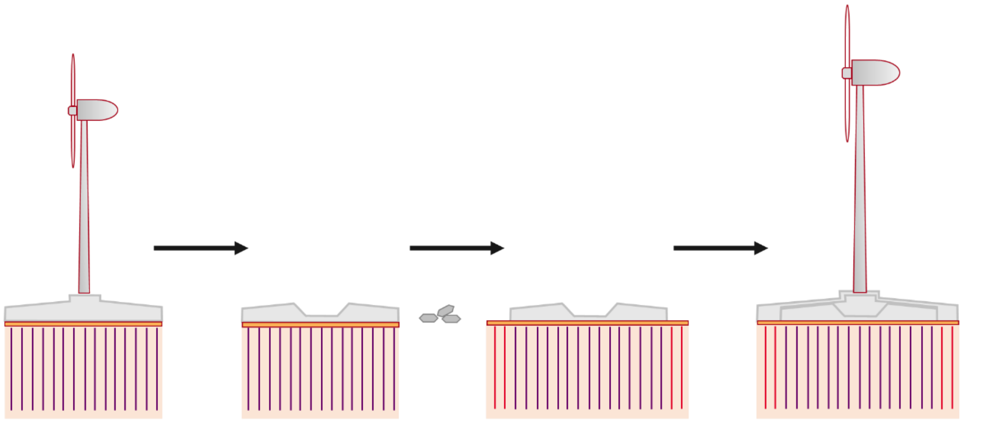

Figure 4: Repowering strategy of FEDRE project

6 GREEN MATERIAL

Cement production significantly contributes to global CO2 emissions, accounting for approximately 7 percent of the world's total emissions. As construction activities continue to rise, there is a discernible upward trend in cement consumption. In 2021, cement production was nearly five times higher compared to 1990 levels (Chaudhury, R., et al. 2023). In Section 3, we highlighted that a significant portion of ground improvement techniques doesn't necessitate extensive use of construction materials. In ground improvement techniques such as CMC, the volume of concrete utilized is notably lower compared to alternative approaches. Additionally, CMC typically maintains a compressive strength ranging from 15 to 25 MPa, in contrast to the higher strength often associated with piling resulting by a significant reducing cement usage and therefore global CO2 emissions.

Much of the sustainability-focused research in ground improvement highlights the critical importance of sustainable practices, especially through the use of environmentally friendly materials. One notable measure to achieve sustainability goals in the construction sector is the use of "green concrete," which is specifically designed to reduce CO2 emissions. This involves a multi-faceted approach that includes alternative materials and innovative mixes, ultimately reducing the overall cement content. However, it is important to be aware that the widespread adoption of low-carbon cement faces significant regulatory challenges, particularly in complying with international and national concrete standards.

A compelling example showcasing the promise of green concrete emerges from the applied research of "exegy." Their development of low carbon concrete has proven capable of achieving a remarkable 70% reduction in CO2 emissions when compared to traditional concrete. This significant advancement is substantiated by calculations depicted in the following (Figure 5), which illustrates equivalent CO2 emissions based on European standards (EN 206/CN), contingent upon the concrete's strength characteristics.

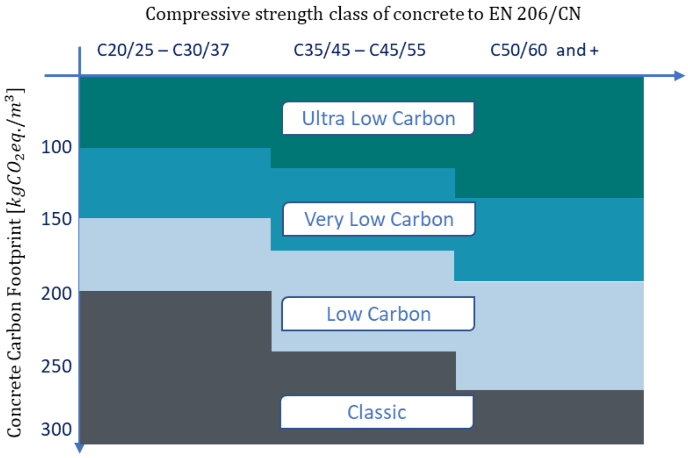

Figure 5: Exegy Standards as a function of CO_2 equivalent emissions of the compressive resistance of the concrete measured at 28 days

7 CONCLUSIONS

In light of the construction industry's daunting energy consumption and resulting CO_2 emissions, the global community is taking proactive measures through various initiatives and dynamic frameworks. Among the numerous sustainability models, those that address the triple bottom line - environmental, economic and social - have emerged as focal points for recent research and are steering the industry towards a more sustainable path. This paper provided insight into the various sustainability models and ongoing research efforts while proposing a streamlined sustainable approach based on fundamental sustainability principles. It also highlighted the inherent sustainability of soil improvement techniques and showed their potential compared to alternative methods. In particular, Controlled Modulus Columns (CMC) were highlighted as a linchpin for the successful reuse of structures in wind turbine repowering to ensure sustainable green energy development. These efforts are not only promising for the wind energy sector but also can serve as a catalyst for other industries, demonstrating that foundation reuse can be a viable option when structures are due for replacement.

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STUDY ON SHEAR RESPONSE OF BIOPOLYMER-MICP TREATED SAND-STEEL INTERFACES

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ABSTRACT

This paper explores the influence of introducing a natural biopolymer, gum of Prunus scoparia (P. scoparia), to Microbially Induced Calcite Precipitation (MICP) treated soil-steel interfaces. The conventional MICP method, involving low-rate injection of cementation solutions into the soil, faces limitations in terms of cost and practical applicability at a field scale. To address this, the natural biopolymer is incorporated into the MICP process, enabling simultaneous application of the cementation solution and gum without controlled injection rates. Through a series of modified direct shear tests, the study investigates the impact of the biopolymer addition to the cementation solution and its potential to reduce the dependency of shear strength parameters on the cementation solution injection rate in treated sand-steel interfaces. The results demonstrate a significant enhancement in shear strength when the biopolymer is introduced into the MICP-treated soil-steel interfaces, independent of the cementation solution's application rate. This innovative approach holds promise for achieving more efficient soil stabilization compared to the traditional MICP method.

1 INTRODUCTION

Indeed, the performance of various geotechnical structures such as floating piles, retaining walls, reinforced slopes, and embankments relies significantly on the response of the soil-steel interface. Therefore, enhancing the strength characteristics of the soil-steel interface is crucial in numerous geotechnical engineering projects. This importance underscores the potential value of adopting practical and environmentally friendly approaches like Microbially Induced Calcium Carbonate Precipitation (MICP) as an alternative to conventional soil improvement techniques, including cement-based and chemical grouting ground improvement methods.

The microbially induced calcite precipitation (MICP) method offers an eco-friendly approach to strengthen the soil-steel interface. It serves as an alternative to traditional stabilization methods such as cement-based and chemical grouting. The MICP method employs urease enzymes to catalyze urea hydrolysis, accelerating the chemical reaction by up to 1000 times compared to uncatalyzed rates. This results in the precipitation of calcium carbonate ($CaCo_3$) within soil voids and on particle surfaces. Various bacterial species, including Sporosarcina pasteurii, Bacillus megaterium, and Bacillus subtilis, serve as biological catalysts in the bio-cementation process.

The bio-cementation process involves specific chemical reactions illustrated in Figure 1 for a granular soil-steel interface. To initiate the process, urea and a calcium source (calcium chloride) are introduced to the soil medium using a cementation solution. Additionally, bacteria are added to the soil medium through a bacterial solution. The MICP process results in the creation of calcium carbonate, also known as bio-cement, that forms between the soil particles. Earlier research has demonstrated that the formation of bio-cement between the soil particles and steel within the soil-steel interface plays a crucial role in enhancing the shear strength properties at that interface (Bak et al., 2021; Sharma and Satyam, 2021).

Earlier research on the MICP technique indicates that applying cementation solution at rates below 10-40 cm/hr leads to the creation of more uniform and superior-quality bio-cemented soil samples. These samples are characterized by well-developed calcite crystals (Whiffin et al., 2007; Mortensen et al., 2011; Cheng et al., 2013). Consequently, injecting cementation solution into one cubic meter of sandy soil could take between 2.5 to 10 hours, presenting challenges related to equipment and labor costs, as well as the implementation time for practical MICP soil stabilization projects.

This study focuses on evaluating the potential of Zedo gum, a water-absorbent biopolymer (Rahimi and Abbasi, 2012), as an additive to the MICP method. The utilization of Zedo gum in this study is motivated by its exceptional water-absorption capacity, as demonstrated by Rahimi and Abbasi (2012), which enables it to effectively address concerns related to calcium solution applying rates impacting the strength parameters of MICP-cemented soil samples. Zedo gum, harvested from Prunus scoparia Spach in Fars province in Iran, forms biofilm networks when combined with cementation solution simplifing the application process by eliminating the need for precise injection rate monitoring. Therefore, it leads to time savings and can improve efficiency in implementing MICP method for soil stabilization projects. Additionally, Zedo gum exhibits soil-stabilizing properties, as seen in previous studies (DABESTANI et al. 2018; Khalesi et al., 2012). While other natural biopolymers (like, Arabic and Xanthan gum) and synthetic materials (like, polyacrylamides) could be considered, the unique combination of properties offered by Zedo gum and MICP reactions makes it a compelling choice for this study.

The objective is to mitigate the impact of cementation solution application rate and enhance the practicality of the MICP method for interface improvement. To accomplish this goal, a series of modified direct shear tests were conducted on soil-steel interfaces, and the shear strength parameters of the soil-steel interface, namely the friction angle and cohesion, are estimated. These parameters offer valuable insights into the effects of the MICP method on the shear response of the soil-steel interface.

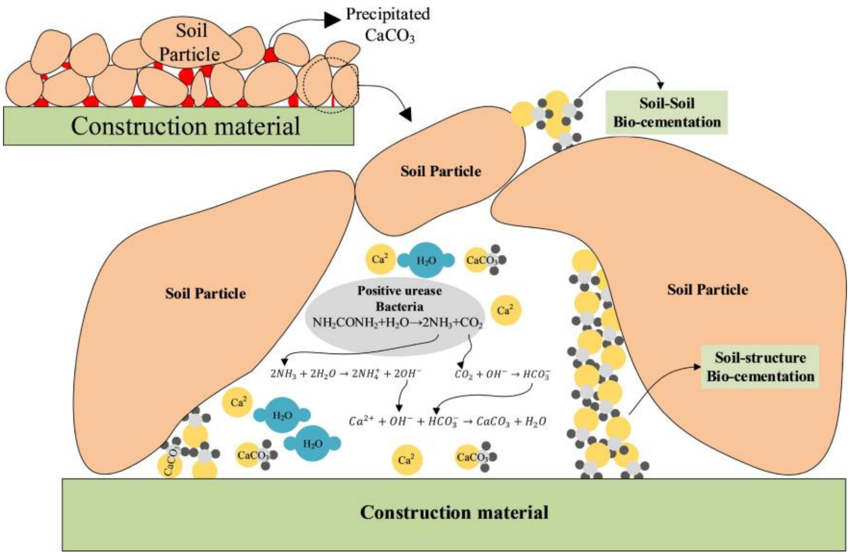


Figure 1: Bio-cementation process in the soil-steel interface (Bak et al., 2021)

2 MATERIAL AND METHODS

This section will initially present the characteristics of the soil and soil-steel interface, followed by a discussion on the preparation of cementation solutions, bacterial solutions, and gum solutions.

2.1 SOIL

In all the tests, a poorly graded sand (SP) was employed. The gradation curve of the soil can be observed in Figure 2. All samples were prepared at a relative density of 30%. Minimum Unit Weight and specific gravity of the soil was 14.2 kN/m^3 and 2.76, respectively.

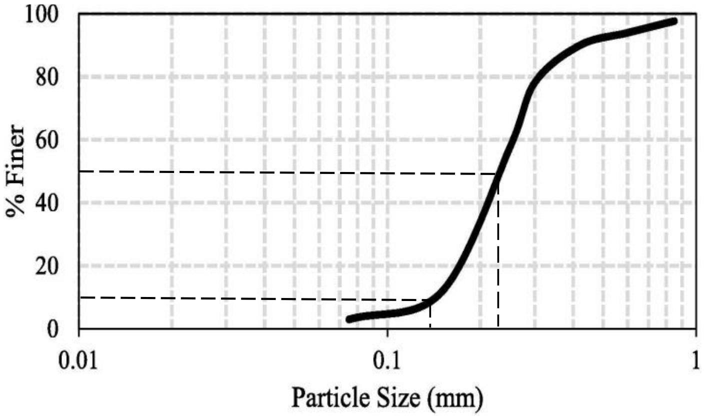


Figure 2: Grain size distribution of the SP sand

2.2 INTERFACE MATERIALS

In this study, a 4 mm thick ST37 steel plate was utilized to examine the interface shear strength. Figure 3 displays the surface profile of the steel plate, which was measured using the Mitutoyo SURFTEST SJ-210 portable surface roughness tester. In Figure 3, The parameter denoting the maximum peak-to-valley distance of the surface profile is referred to as R_{max}. Uesugi and Kishida (1986) introduced the normalized surface roughness (R_n) as a parameter to quantify the roughness of the soil-interface system. It involves normalizing R_{max} against the mean grain size of the interface soil, as represented by the following equation:

$$R_n = {R_{max}}/{D_{50}} \quad (1)$$

The normalized surface roughness (R_n) has been widely employed by scholars as a reliable measure to assess the roughness of soil-steel interfaces (Mortazavi Bak et al., 2021; Su et al., 2018). Previous studies have shown that practical R_n values often exceed 0.05. A soil-construction material interface is typically considered rough when R_n is greater than 0.1-0.15, depending on particle size and shape.

In this study, the soil's D_{50} value was 0.21 mm, and the soil-steel interface's R_{max} was 0.05 mm, resulting in R_n > 4, which falls well within the range of rough interface systems.

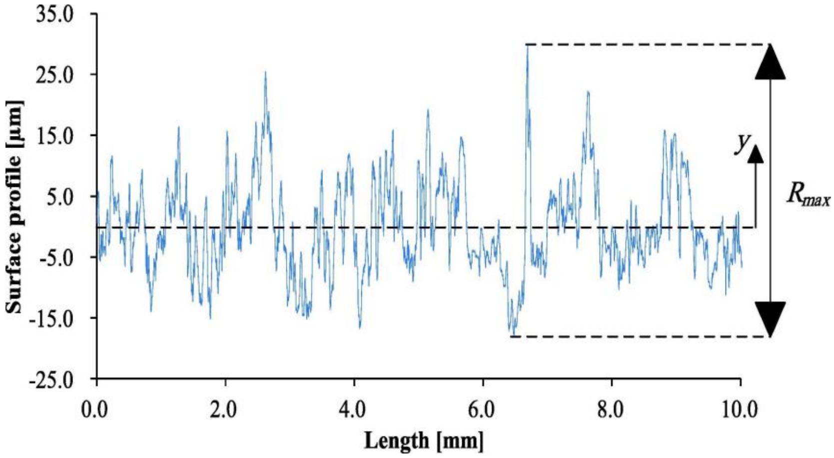


Figure 3: The surface profile of the steel plate sample (Bak et al., 2021)

2.3 PREPARATION OF CEMENTATIN, BACTERIAL, AND GUM SOLUTIONS

In this study, the *S. pasteurii* strain (PTCC 1645) was employed as a suitable urease bacteria for MICP method (Mukherjee et al., 2019; Khaleghi and Rowshanzamir, 2019; Ng et al., 2012; Whiffin, 2004), and the average bacterial population during the stationary phase was 10^9 per ml. The pH was initially set to 7 in an enriched Nutrient Broth (NB) culture medium with urea for MICP, following previous research (Bak et al., 2021). After cultivation, bacteria were collected, washed, and reintroduced into a fresh medium (Wu et al., 2019; Stocks-Fischer et al., 1999; Bak et al., 2021). Equimolar Calcium Solutions with 1 mol/lit concentration of urea and calcium chloride were prepared, following previous studies (Al Qabany et al., 2012; Bak et al., 2021). To prepare the gum solutions, Zedo gum was prepared by grinding, sterilizing, and mixing with water at concentrations of 10mg/ml using a magnetic mixer (60°C, 150 rpm) for two hours.

3 XPERIMENTAL PROGRAM

3.1 MODIFED DIRECT SHEAR TEST

A large-scale direct shear test apparatus, with the sample size of 300 mm×300 mm, was utilized to investigate the soil-steel interface response under the influence of bio-cementation, as depicted in Figure 4. Both standard direct shear tests and modified interface direct shear tests were conducted following ASTM D5321 and ASTM 3080 standards, respectively. In the soil-steel interface tests, a five-legged steel plate (illustrated in Figure 4) was substituted for the soil in the lower part of the box.

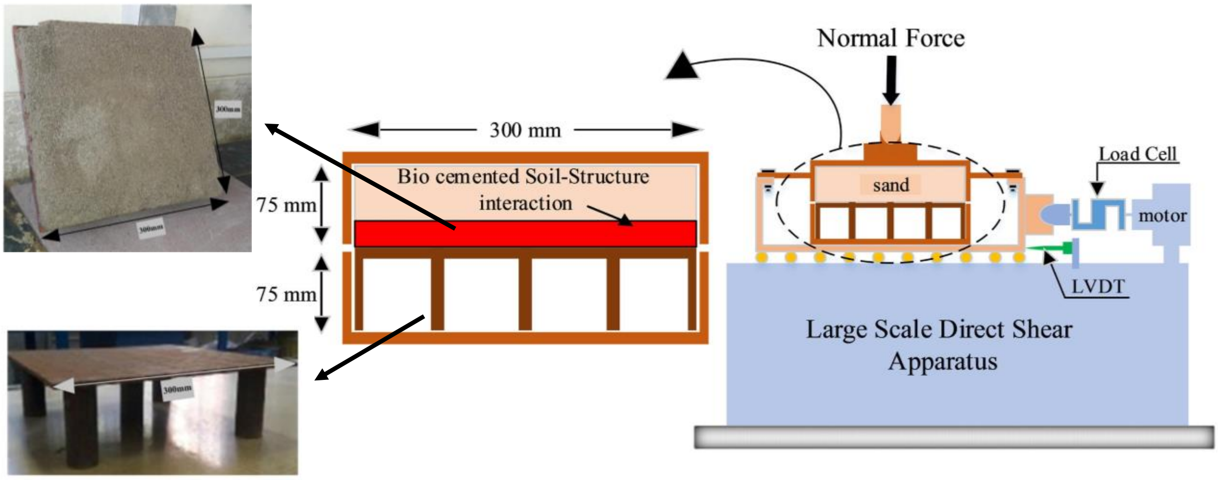


Figure 4: Modifeid direct shear box

3.2 SAMPLE PREPARATION AND TEST PROGRAM

In this study, the soil was placed onto the steel interface, carefully adjusting the desired void ratio and thickness depending on the type of the sample. The next step involved spraying the bacterial solution onto the soil-steel interface sample with a volume equal to the soil void volume (V_v) using hand sprayer. After allowing an hour of rest, a mixture of cementation solution and gum solution was sprayed onto the soil specimen. For samples without gum solution, only cementation solution was sprayed. The spraying volumes were set to V_v for cementation solutions and 0.25 V_v for gum solution, determined according to the experimental plan. After a retention time of 24 hours, another mixture of gum solution and cementation solution was sprayed onto the soil-steel interface systems, with volumes of 0.5 V_v and 0.125 V_v, respectively. The samples were then cured for 14 days before conducting the shear tests. To eliminate the potential impact of suction on the shear strength results, the G-MICP-treated soil-steel interface systems were dried in an oven.

In the current study, 9 different test conditions presented in Table 1 are considered. Table 1 categorizes the samples as U (untreated), M (MICP treated), and BM (biopolymer-MICP treated). The numbers following the letters represent the respective normal stresses applied during the tests. For example, BM100 means a biopolymer-MICP treated sample subjected to a normal stress of 100 kPa. In each test condition, a total of three samples were prepared, and each sample was subjected to a constant normal stress of 50 kPa, 100 kPa, and 150 kPa.

Table 1. Experimental program of the soil-steel interfaces

Set	Experiment	Concentration ratio of urea (mol/litre)	Concentration ratio of $Cacl_2$ (mol/litre)	Spraying rate ($cm^3/h/cm^2$)	Gum Concentration (mg/ml)	Normal Stress (kPa)
A	U50	-	-	-	-	50
	U100	-	-	-	-	100
	U150	-	-	-	-	150
B	M50	1	1	5	-	50
	M100	1	1	5	-	100
	M150	1	1	5	-	150
C	BM50	1	1	10	10	50
	BM100	1	1	10	10	100
	BM150	1	1	10	10	150

3.3 SOIL STRENGTH PARAMETERS

In Mohr-Coulomb shear failure criterion, the interfacial shear strength parameters, denoted by δ and c_m, represent the interfacial friction angle and cohesion, respectively. These parameters have been widely used previously to characterize the shear strength behaviour of soil-construction material interfaces. The following equation defined the relationship between the shear stress and normal stress at the modified direct shear test:

$$\tau_{pm} = c_m + \sigma_{nm} \tan(\delta) \quad (2)$$

where τ_{pm} and σ_{nm} are the peak shear stress and the normal stress (i.e., 50, 100, and 150 kPa) in the direct shear tests or modified direct shear tests.

The interface efficiency factor (IEF) is also defined as the ratio of the peak shear stress of the treated sample to the peak shear stress of the untreated sample at a specific normal stress (Bak et al., 2021). Accordingly, IEF can be calculated by the following formula:

$$IEF = \tau^{t}_{pm} / \tau^{u}_{pm} = (c^{t}_{m} + \sigma_{nm} \tan\ (\delta^{t})) / (c^{u}_{m} + \sigma_{nm} \tan\ (\delta^{u})) \quad (3)$$

where superscripts t and u stand for treated and untreated, respectively. Based on Equation 3, a more successful bio-cementation process yields a higher IEF. This parameter is employed in this study to compare the test results in terms of the gain in the interface shear strength due to bio-cementation.

4 RESULTS AND DISCUTION

By utilizing the test results from experiment sets A, B, and C, it is possible to determine the friction angle and cohesion of the untreated samples, MICP treated samples, and biopolymer-treated samples. The subsequent sections will focus on discussing the influence of MICP treatment and the addition of gum on the strength characteristics of the soil-interface. Table 2 summarises the results of the interfacial shear strength parameters of samples.

Table 2. Interfacial shear strength parameters of samples

Sample	Interfacial friction angle (°)	Cohesion (kPa)
untreated soil-steel interface	24.8	0
MICP treated soil-steel interface	42.3	112.5
Biopolymer-MICP treated soil-steel interface	42.6	120.6

4.1 FRICITON ANGLE

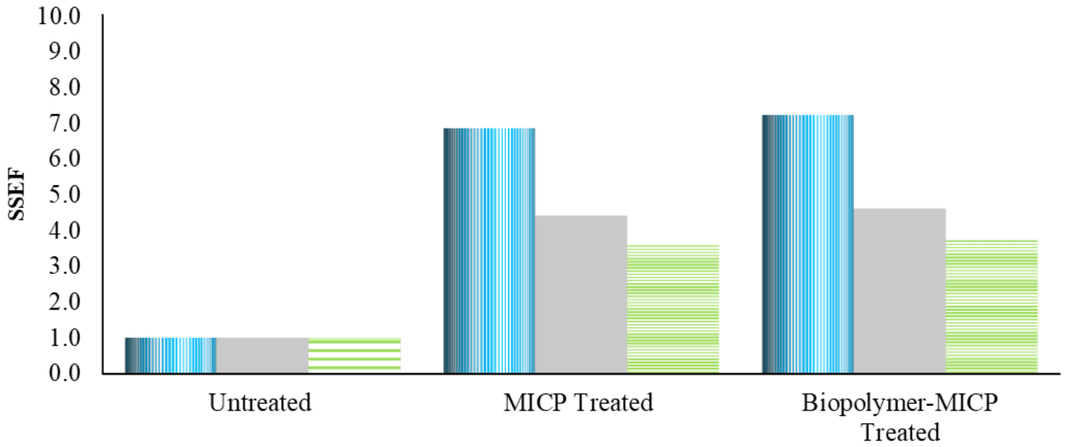
Analysing the test results based on the Mohr-Coulomb shear failure criterion and using Equation 2 reveals that the friction angle of the untreated soil-steel interface was measured at 24.8°. However, after the MICP treatment, the peak friction angle increased to 42.3°, and with the biopolymer-MICP treatment, it further increased to 42.6°. This indicates that the addition of a very low concentration gum solution allows for a twofold increase in spraying rate, while still achieving a higher friction angle compared to the MICP treated samples.

4.2 COHESION

Upon analysing the test results as described in the previous section, it was found that the cohesion values for the MICP treated samples and biopolymer-MICP treated samples were 112.5 kPa and 120.6 kPa, respectively. These findings were consistent with the results of the friction angle, indicating that the addition of a very low concentration of Zedo solution can reduce the influence of spraying rate on the shear strength parameters.

4.3 INTERFACE ENHANCEMENT FACTOR

In this section, the results of the friction angles and cohesions are utilised to further investigate the interface shear strength of samples under different normal stresses in terms of interface enhancement factor using Equation 3. As shown in Figure 5, biopolymer-MICP treated samples exhibit higher interface enhancement factors (IEF) at various stress levels, which aligns with the findings from previous sections. Additionally, Figure 5 demonstrates that the treatment's effect is more pronounced at lower stress levels, consistent with previous studies (Bak et al., 2021).



	Untreated	MICP Treated	Biopolymer-MICP Treated
50 kPa	1.0	6.8	7.2
100 kPa	1.0	4.4	4.6
150 kPa	1.0	3.6	3.7

Figure 5: IEF of the samples at different stress levels

5 CONCLUSIONS AND RECOMMENDATIONS

The impact of adding gum to the MICP treated soil-steel interfaces has been explored. To this end, a series of modified direct shear tests were conducted, and the results were analysed based on the Mohr-Coulomb shear failure criterion. The findings indicated that the biopolymer-MICP treated samples exhibited higher friction angle and cohesion compared to the MICP treated samples, even with lower spraying rates. Moreover, by incorporating very low concentration gum solutions and doubling the spraying rate, the interface enhancement factor (IEF) increased from 6.8 to 7.2, rendering the MICP method more practical and promising for engineering applications.

The results of this study indicated that the incorporation of gum into the bio-cementation process reduces the sensitivity of the shear strength parameters to the spraying rate variations at the interfaces. Using higher spraying rates is more feasible for field applications of the MICP method, enhancing practicality.

While this study demonstrated the efficacy of biopolymer-MICP treatment, further investigations are needed to thoroughly examine the type of the crystals precipitated between the soil particles, durability and CO_2 emission reductions of this method in comparison to conventional methods, and influence of environmental conditions, soil type, and other relevant parameters to gain a comprehensive understanding of the process.

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