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CIVIL ENGINEERING
Isivili Enjiniyering
April 2017 Vol 25 No 3

Civilution
2030 starts with you

Focus on: Geotechnical Engineering

- Southern Cape Landslip
- Upgrading the Kranspoort Pass

Profile: Dr Phil Paige-Green

PASSION
FOR PROGRESS

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The Don

A good name is more desirable than great riches …
(Proverbs 22:1)

On the morning of Ahmed Kathrada's death, the renowned and respected cartoonist Nanda Sooben took to social media asking, "Are there any good men left?"

After the recent cabinet reshuffle, I asked our members if SAICE should take a stand. Some of our members have encouraged SAICE to engage. SAICE's young members, particularly, wish SAICE to be heard, and seen to be heard on the matter.

Other members are quiet – and I respect that, too. SAICE has always cherished the complexity of views that emanate from our diverse membership. This is the brilliance of our own democracy. But I must make the point that Elie Wiesel made – indifference, while it is tempting, is a peril. Wiesel argues that, because it benefits the aggressor and not the victim, indifference is a friend of the enemy.

At the recent ICE Conference in Cape Town (April 2017), Yunus Ballim PrEng, Vice Chancellor of Sol Plaatje University in Kimberley, and professor of civil engineering at Wits University, on a platform with distinguished colleagues Sundran Naicker PrEng and Paul Jowitt CEng, articulated two gems that appealed to my civil engineering sense of social justice. He said:

- To be a civil engineer, is to be fundamentally engaged in critical matters of the human condition; and
- For civil engineers to avoid politics – to not be involved in politics – is flawed and imaginary. Civil engineers must intervene in places of power and spaces of powerlessness.

Civilution expressly requires of us to be honest with introspection. In the malaise of our country's past, SAICE practised great circumspection when sharing views outside of cold concrete civil engineering. At the turn of the millennium, some of our members from previously disadvantaged communities reluctantly joined SAICE, because SAICE was "deafeningly quiet" during the apartheid years, when they needed us most. This is why many black civil engineering practitioners still claim that SAICE is a "white" organisation. The culture of SAICE, supported by our membership statistics, shows otherwise. But we shouldn't miss the point. SAICE should not repeat its mistakes, as we will be judged severely for it in future, in both moral and metaphysical terms.

I attended the funeral of Donald Macleod PrEng, who was City Engineer of Durban from 1976 to 1992. Millions of people enjoy safe sanitation in Durban because of Don's leadership. At the memorial I noticed the humility and understated elegance of the full life of a good man. In his SAICE presidential address in 1987, he said, "We should never see the fruits of technology as being of greater importance than people. Our respect for the dignity and immeasurable value of the human being should always be upheld." He was known as a leftist in the nationalist climate of the day. As a white civil engineer, he was known for rescuing black people during the Cator Manor uprising, and delivering sanitation to black communities in a time when policies and nationalist establishment dictated otherwise.

Having worked with his son, Neil Macleod PrEng, at SAICE, the Macleod name is in the company of those bastions of social justice mentioned in this article – so, too, is the name of every civil engineering practitioner who abides by the traditions and tenets of this incredible profession.

I am aware that SAICE is a nonpartisan, impartial and unprejudiced voice for civil engineering professionals. Our objectives are the growth and development of our members, and the promotion of the science and practice of civil engineering and the advancement of the civil engineering profession. Ahmed Kathrada would have agreed with us and then said, "… I express the hope that you will choose the correct way."

So what is the correct way? My personal answer to the icon would be along these lines:

"Our involvement is unattached to any individual, political party or schisms in party politics. It is principally associated with good governance and the role of state in creating conditions for democratic process, and social justice in South Africa. Civil engineering serves all South Africans. Recent instances of dysfunctional and unaccountable behaviour in parliament, as well as unclear reasons for ministerial appointments, cause concern about our government's ability to properly respond to the development, infrastructure and socio-economic well-being of South Africa. I am a civil engineer. I protest for the South Africa I love, because I believe in its resilience."

With sword in one hand and pen in the other, my answer to Nanda Sooben is, "Yes! There are still good men and women left amongst 52 million South Africans. We have 13 000 of them – we are civil engineering professionals." ◻

Isivili Enjiniyering = SiSwati

ON THE COVER

Keller's Franki Africa is known for overcoming challenges, and for delivering cost-effective geotechnical solutions using a wide range of technologies in a host of different ground conditions, as demonstrated in its recent trenchless work on a sewer line in Paarl (the photo shows the treated jacking face holding cobbles and fines in suspension).

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ON THE COVER P7

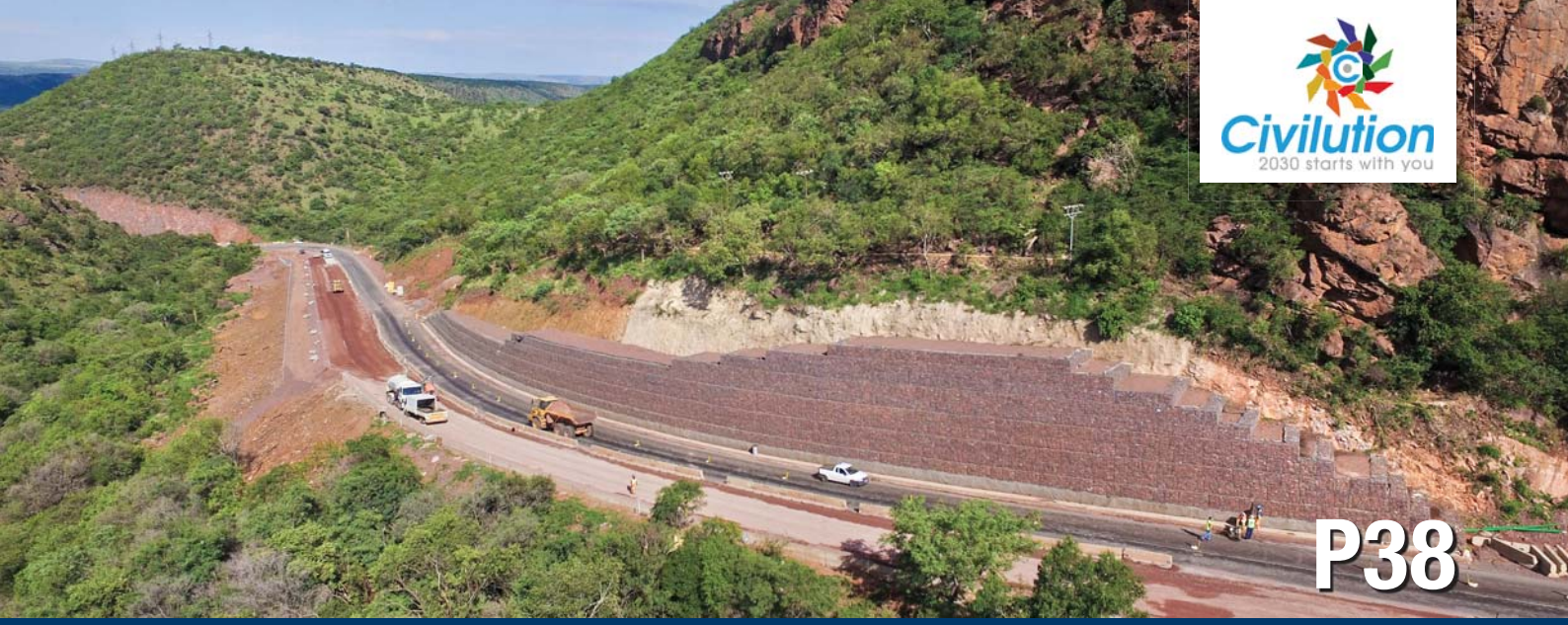
▶ **The 8.5 m shored jacking pit ready to start pushing the jacking shield in a trenchless technology operation on the Paarl bulk sewer line in the Western Cape**

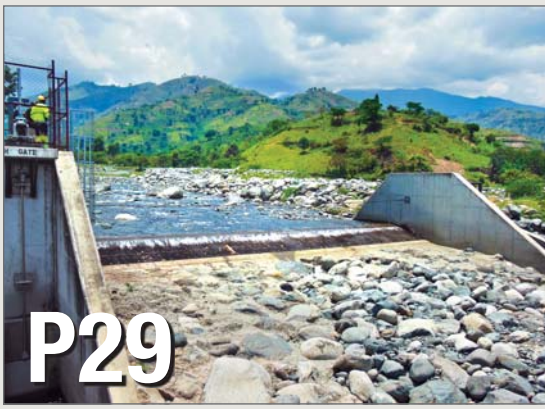
FROM THE CEO'S DESK

ON THE COVER

PROFILE

GEOTECHNICAL ENGINEERING





SAICE AND PROFESSIONAL NEWS

The history of Stefanutti Stocks Geotechnical
- Highlights along the way

2003

In 2003 Stefanutti & Bressan (which was founded in 1971) established a piling division which mainly operated in Durban and along the coast.

2005

Two years later the company made a substantial capital investment into modern equipment and with this equipment it established an independent piling company in Gauteng. Named S&B Geotechnical & Piling to reflect its multidisciplinary geotechnical offering, this new company offered the market piling works, lateral support and consolidation and compaction grouting.

One of its first major contracts in the Gauteng area was for the PFG glass factory in Springs. This project consisted of bulk earthworks, lateral support and piling, and required:

- 160 lateral support piles
- 260 ground anchors
- 100 metre of soil nails
- 1,650m³ of shotcreting to support the nine-metre deep excavation.
- 600 continuous flight auger piles (CFA) load-bearing piles ranging between 500mm to 900mm in diameter, and going to depths of up to 23 metres.

2008

In 2008, after the unveiling of the newly merged Stefanutti Stocks group, Stefanutti Stocks Geotechnical, a division of Stefanutti Stocks (Pty) Ltd, was formed.

In late 2008 Gautrain projects were tendered on and the Bombela Civils Joint Venture awarded several contracts to Stefanutti Stocks Geotechnical. These included:

- Six lateral support sites along the Gautrain line from Frankenwald to Nellmapius Roads, as well as the Allandale cut. On these six lateral support sites 11,365 metres of soils nails were installed and a total of 1,440m³ of shotcrete applied.
- The lateral support for the entrances to the Gautrain Station in Rosebank was constructed. These entrances consisted of two holes, each 60-metre long, nine metres wide and 11,5 metres deep. Both entrances required contiguous piled walls with anchors and shotcrete arches.
- The consolidation grouting of 49 pier positions between the N1 at John Vorster and the N14 at Jean Ave, with each pier position requiring the drilling and grouting of 81 boreholes. A total 151,361 metres were drilled to place 110,983m³ of grout.

2010

Stefanutti Stocks Geotechnical, as the lead partner in a 50/50 joint venture, was awarded the piling to the Kusile Power Station. The in-situ ground conditions at the power station necessitated extensive piling including the casting of some auger in-situ piles up to 25 metre deep. These varied from 800mm to 1200mm in diameter.

At the end of an almost five-year programme, which saw the civil contractors working on various work packages across the power station, the joint venture had installed a total of 8,540 piles, used 777,725m³ of concrete and 9,229 tons of reinforcement, as well as having drilled 12,882 metres for the installation of the piles.

In 2010 Stefanutti Stocks Geotechnical also successfully completed their first major cross-border project in the Tonkolili district in Sierra Leone. The success of this project saw Stefanutti Stocks Geotechnical expand their geographical footprint into Africa

As a leading Southern African geotechnical contractor, Stefanutti Stocks Geotechnical continues to pursue excellence in execution by putting its years of experience across multidisciplinary geotechnical capabilities and services to work to the benefit of its clients.

2012

During this period, an increase in basement development in upmarket areas like Sandton, Menlyn, Brooklyn and Rosebank was experienced.

Some of the basement construction completed, within a relatively short period of time, included:

- Sandton City Repositioning Project
- Menlyn Maine Epsilon Building
- Menlyn Maine Falcon Building
- Menlyn Maine Pegasus Building
- Brooklyn Point Office Block
- Corobay Corner Office Block
- Boardwalk Hotel, Port Elizabeth

2015

Stefanutti Stocks Geotechnical successfully completed the design and supply contract for the deep foundations to the Kazerne Transit-Oriented Development project in Newtown, Johannesburg. This comprised of 25,039m³ of bulk earthworks, 1,750m² of permanent lateral support, 440 structural piles and temporary traffic diversion.

The piling work comprised of 242 Continuous Flight Auger (CFA) piles, installed up to 23 metres deep; 133 polymer-technology drilled piles installed to a depth of up to 30 metres deep; as well as 65 Auger Cast In Situ (ACIS) Soldier piles installed to depths of up to nine metres.

2016

In late 2016 the design, supply and installation of piling to the Xixamba River bridge in Bushbuckridge along the new road from Marijane to Merry Pebbles was completed. Following the geotechnical investigation work, Stefanutti Stocks Geotechnical submitted a design and supply proposal, based on the information from the investigation. This proposal included the installation of 21 temporary cased, cast in-situ piles, varying between 900mm and 750mm in diameter and installed to depths of up to 25 metres.

During the piling operation a total of 220 metres of different classes of rock was drilled through, ranging from R2 rock (3-10Mpa) up to R5 (70-200Mpa). In spite of heavy rain and the resulting wet conditions complicating the piling operation, the Stefanutti Stocks Geotechnical site team still completed the project well ahead of schedule.

Stefanutti Stocks

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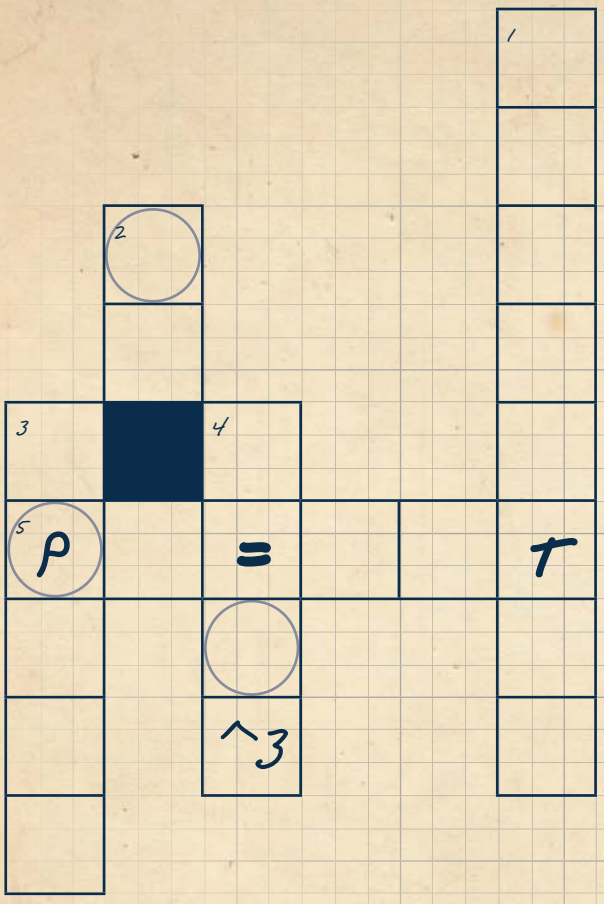

DOWN

1. The formula p=m•v is used to calculate.
2. The SI Unit for Pascals (the derived unit to quantify internal pressure).
3. The letter S in S=d/t is used to notate which scalar quantity?
4. Formula for the volume of a cube.

ACROSS

5. The equation stated by Émile Clapeyron in 1834 as a combination of the empirical Boyle's law, Charles' law and Avogadro's Law commonly known as the ideal gas law.

Franki overcomes challenges on the Paarl bulk sewer line

INTRODUCTION

Keller's Franki Africa has a reputation for being able to deliver cost-effective geotechnical solutions using a wide range of appropriate technologies in a host of different, and often challenging, ground conditions. "We have worked in southern Africa for many decades and have a profound understanding of the different soil conditions and the optimal geotechnical solutions for them," says Franki's Trenchless Technology Manager, Byron Field.

He adds that this knowledge enables the company to be pro-active in solving problems that, on the face of it, sometimes seem insoluble. The Drakenstein Municipality's bulk sewer pipeline is an excellent example of this.

THE CHALLENGE

The path of the sewer pipeline (in the town of Paarl in the Western Cape) includes a stretch of approximately 105 m across Arboretum Road and the N1 highway, followed by a section of around 110 m which runs parallel to the Boschenmeer Golf Estate boundary wall at a depth of –6 and –8 m.

According to Field, the main challenges were the relatively unstable ground conditions, which comprised sands from 0 to –4 m deep, with loose cobbles and boulders from –4 m to –8 m, and a very high water table.

SANRAL also had strict wayleave conditions prohibiting the Drakenstein Municipality from conducting work beneath the N1 unless they were able to prove that every conceivable precaution had been taken to protect the highway and to ensure uninterrupted use.

It was obvious that a trenchless methodology, like pipe jacking, would be required for the new sewer to run under the roads without interrupting traffic. The depth of the pipeline and its proximity to the Boschenmeer Golf Estate boundary wall also made open excavation impractical.

When the ground conditions were analysed, a new challenge was encountered! Field explains: "Firstly, the level of the sewer passed directly through the cobble layer between –4 to –8 m deep, and when pipe jacking is performed through this type of ground it is virtually impossible to prevent collapse of the cobbles during excavation. Secondly, the high water table tended to draw fines from the surrounding ground towards the jacking shield. Both of these conditions could have led to over-excavation resulting in ground level settlement."

Jet grouting rig working on the Paarl bulk sewer line alongside the N1

Treated ground at jacking face

Exposed trial jet grout column clearly showing dense cobbles that are bound together after treatment

FRANKI'S PROACTIVE PROPOSAL

Franki approached the Drakenstein Municipality with a proposal to treat the ground beneath Arboretum Road and the N1, as well as alongside the Boschenmeer Golf Estate boundary wall.

The proposal entailed jet grouting – which involves the mixing and partial replacement of the in-situ soil with cement slurry – to consolidate the in-situ ground condition along the sewer centreline and between the depths of –4 to –9 m, and to then install a pipe jack through the treated ground. "The treatment of the ground would prevent collapse of the sand and cobbles during pipe jack excavation and would reduce the ingress of water to manageable levels," Field says.

He adds that jet grouting was Franki's preferred method of treatment, as high-pressure jetting can be used to consolidate in-situ ground at exact levels, and can provide up to 2.5 m diameter columns with only an 80 mm drill stem.

The municipality's design team included the proposed solution in the tender document for this phase of the works and, in August 2016, Franki was appointed by the main contractor, Vakala Construction, to carry out the specialist geotechnical work.

THE RESULT

Field says that the jet grouting went according to plan and was carried out with zero impact on traffic. "In addition, once the jet grouting had been completed and the site cleared, there was no remaining evidence at ground level that the ground beneath had been treated.

"The entire pipe jacking operation went smoothly, with the ground treatment working better than even our highest expectations."

The sub-contract work was completed by Franki on time (February 2017) and within budget.

FRANKI – MORE THAN JUST PILES

Franki is renowned for its geotechnical solutions using an array of different piles, including driven tube piles, precast piles, auger piles, full displacement screwpiles, rotapiles, micropiles, the famous Frankipile (driven cast-in-situ pile) and many more. It is also well-known for its soil improvement systems, including dynamic compaction, deep soil mixing, accelerated consolidation, and of course jet grouting as discussed above.

Franki's skills in trenchless technology are just as impressive. For more than 30 years it has successfully been providing pipe jacking and other trenchless technologies – augering, thrust boring and large-diameter case boring – to a wide range of clients in southern Africa.

Trenchless technology is a 'family' of methods, materials and equipment capable of being used for the installation, replacement or rehabilitation of existing underground infrastructure with minimal disruption to surface traffic, business and other activities. It is, therefore, often the most cost-effective solution.

Pipe jacking, an integral part of this 'family', is a technique for installing underground pipelines, ducts and culverts. Powerful hydraulic jacks are used to push specially designed pipes through the ground behind a shield at the same time as excavation is taking place within the shield. The method provides a flexible, structural, watertight, finished pipeline as the tunnel is excavated.

CONCLUSION

By being part of the Keller Group, Franki's leadership in the geotechnical space in southern Africa has been significantly enhanced. Keller is the world's largest independent geotechnical engineering contractor, offering Franki significant advantages, such as access to a wide range of innovative technologies, state-of-the-art machinery and a wealth of geotechnical intellectual property and experience.

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Loving his job because it's meaningful

Dr Phil Paige-Green, the both-feet-on-the-ground 2016 recipient of SAICE's Geotechnical Gold Medal, received the news of his nomination for this award while working near Xai Xai in the Gaza Province of Mozambique – after borrowing a modem to download emails so as to avoid too many when he got back to civilisation.

Dr Phil Paige-Green, 2016 recipient of SAICE's Geotechnical Gold Medal

"While I had the modem on loan I had to respond to that email by either accepting or declining the award. I thought about past recipients such as Jere Jennings, Geoff Blight, Tony Williams, Tony Brink, Fritz Wagener, Peter Day and others, and wondered if I, an engineering geologist, should ever be placed on the same pedestal as any of them. I didn't really think so, but agreed to accept the medal before the committee changed their minds! I must say that I feel *really* honoured and humbled to have been recognised in this way."

EARLY LOVE AFFAIR WITH STONES

When Phil was around six or seven years old, while living in a small mining town in Swaziland, he was given a few stones by an old woman on the mine who was moving house. From then on (with some help from the mine geologist) he was always going to be a geologist. Living on some of the world's oldest ultramafic rocks belonging to the 3.5 billion years old Onverwacht Group of the famed Barberton Super Group probably also helped in developing his geological disposition.

"Behind our house on the mine was a 4 to 5 m high cutting in clay – I would now probably call it a 'slightly moist, red mottled orange, firm to stiff, fissured, silty clay, residual basalt/greenstone' – in which I had excavated many metres of roads, half tunnels, full tunnels and underground cavities for my collection of dinky cars. Little did I know then that I would spend more than 40 years of my life involved in these on a much larger scale. I also used to gaze at the landslides in our backyard each time it rained, wondering why this happened, and where all of my structures and roads had gone – obviously non-climate resilient! I knew that the rain caused it, but didn't know why at that stage."

A gift of a few stones when Phil was small inspired a lifelong passion for geology

GAINING AN UNDERSTANDING OF STONES AND SOILS

After moving to Durban and completing high school, Phil did a BSc in Geology at the University of Natal, including an introductory course in engineering geology from Rodney Maud, which resulted in him topping up his degree with a Master's in Engineering Geology, based on Rodney's fascinating tales of life as an engineering geologist.

This was the start of his training and mentorship under such influential people as Ken Knight (Phil's lecturer in soil mechanics and co-supervisor of his MSc), Rodney Maud (lecturer and examiner of his MSc), and others. At the time (1975) he had the opportunity to attend his first of seven Regional African Conferences on SMFE (Soil Mechanics and Foundation Engineering) in Durban, where he met numerous people who later influenced his career significantly.

The damage to the approach fill of this bridge in Mozambique during the 2013 floods illustrates the overlap between engineering geology and geotechnical engineering

While at the University of Natal, Phil was offered a position at the then National Institute of Road Research at the CSIR, where he started in 1976, and remained until his retirement from the CSIR in 2013.

"I consider myself lucky in my career. As Isaac Newton said way back in 1676: 'If I have seen further, it is by standing on the shoulders of giants'. I have managed to meet and work with many of the giants in the geotechnical field, all of whom have helped me, a relative dwarf, to see further. In fact, there are only three or four names on the list of the past 32 Gold Medal recipients that I never got to meet. Many of the remainder I have worked closely with over the years – these include such giants as Tony Williams, Tony Brink, Gary Jones, Frank Netterberg, Hartmut Weinert, Peter Day and many others. I am also happy to see that I am the fourth engineering geologist to be honoured with this prestigious engineering award, the others being Tony Brink, Frank Netterberg and Hartmut Weinert, all of whom I have worked with over the years, and coincidentally all of whom had spent at least part of their careers at the CSIR."

FIRST GEOTECHNICAL TASK

When he started at the CSIR, Gary Jones was his first boss, and one of his first geotechnical jobs was the proposed Wonderfontein section of the N4. Gary told Phil to go and profile the sites of two embankment fills and a bridge foundation, and do some CPT profiles using the newly introduced equipment (before piezocones).

"I did the field work, went back to the office and sent the field sheets through to Gary. He called me in and told me that I had only done half a job, and that I needed to analyse them as well and see if the preliminary designs were adequate. I didn't have a clue what to do and so, after a quick course by Gary in applied foundation engineering, I carried on and did it. It transpired that the materials were very weak, fissured, residual clays, and one of the embankments and the bridge foundations would probably have failed as designed originally – I am pleased to say they are still there today!"

TURNING A DILEMMA INTO MEANINGFUL WORK

The theme of Phil's PhD thesis in the mid-80s (University of Pretoria) was, *The influence of geotechnical properties on the performance of gravel-wearing course materials*. "With my PhD came the realisation that I had a bit of a dilemma – was I doing the work of a geologist or an engineer? The material selection and performance modelling of unpaved road behaviour led onto the design, which was suddenly an engineering issue more than engineering geology, but I knew I couldn't be an engineer, although I did join SAICE at that time as a visitor."

As it turned out, Phil moved into materials investigation for low-volume paved roads and then the design of these, which is what still keeps him busy today as an independent consultant.

"The overlap between engineering geology and geotechnical engineering was nowhere more apparent to me than when I recently looked at bridges which had lost their approach fills during the 2013 floods in the Gaza Province in Mozambique. The embankment strength is a function of the geological/geotechnical properties, as well as of the construction and design. The erosion is a function of the geomorphology, rainfall and design. The failure is thus a combination of geology, geotechnical, geomorphological and engineering inputs, together with the one thing we can't control – the climate. My career has re-treaded my geological background into a much wider horizon overlapping geotechnical and road engineering. I like to think I know enough about all of these issues now to pull them into one composite solution."

And indeed, Phil's life-work has culminated in him becoming a specialist in low-volume roads. "In the past there were no design

I have managed to meet and work with many of the giants in the geotechnical field, all of whom have helped me, a relative dwarf, to see further.

Dr Phil Paige-Green with his wife Pam, who he has been married to happily for 38 years, and their children Timothy and Alexandra

methods specific to low-volume roads. Conventional road design methods were simply downscaled to construct low-volume roads, but we cannot afford that any longer, particularly in light of the fact that at least 75% of our roads are low-volume. Our approach has therefore been to find other ways of designing, using cheaper materials, leaving out whole layers where possible, and so forth, resulting in many more lengths of road for the same amount of money."

Phil revels in the fact that this approach has the potential to uplift the whole of Sub-Saharan Africa. His work in a number of African countries, and also currently in India, entails establishing this design philosophy. In India, Phil and his team are designing 5 000 km of low-volume roads. If one considers that India has approximately 170 000 villages which do not have road access at all, it puts the extent of the need into perspective. In Africa (particularly in Ethiopia, Tanzania, Ghana, Zambia, Malawi and Mozambique) Phil has prepared (or is still involved in) manuals on how to design roads using these methods. The irony is that this same problem exists in rural South Africa, but the available expertise is not fully utilised here.

GLOBAL FOOTPRINT

During his interesting and varied career Phil has worked on every continent (36 countries) except Antarctica, but he says that the lack of roads will not stop him from still going there, too! One of his major growing experiences was spending two years in the Middle East Gulf region in the mid-1990s, based in the Sultanate of Oman, where he was the only engineering geologist/pavement person in the area. Hence he was called in to look at all sorts of problems – slope instability, construction and stabilisation problems, salt damage problems, settlement of buildings, and even the review of a freeway design in Pakistan.

Over the years Phil has also developed a working understanding of languages as diverse as Arabic, Italian, Zulu, French, Portuguese and Afrikaans, that is apart from his native English. Being able to deal with language barriers in the work environment, even if on a limited scale only, has stood Phil in good stead on many projects.

AWARDS AND RECOGNITION

The SAICE Geotechnical Gold Medal is undoubtedly the most prestigious recognition of Phil's work. He received many other awards as well, of which the following are very special to him:

- 1998: Joint recipient of the ATC Award for the best paper presented at the annual Transportation Convention with Dr Frank Netterberg for their paper titled *Wearing course materials for unpaved roads in southern Africa: A review*
- 2000: George Dehlen Award for Excellent Mentorship
- 2008: JD Roberts Research Award

GIVING BACK

The past few years have been a period in Phil's life where he has tried to give something back, his motto being Albert Einstein's saying "Try not to become a man of success; rather become a man of value", the reasoning being that a man of value will give more than he receives. And indeed, Phil has supervised

At least 75% of South Africa's roads are low-volume, necessitating more economical approaches to road design and construction

six PhDs and numerous Master's theses, and lectured for many years at various universities in South Africa, which led to his appointment as Extraordinary Professor in the Faculty of Engineering and the Built Environment at the Tshwane University of Technology three years ago, where he taught geology for engineers, geomechanics, construction materials and concrete technology. Phil found this very rewarding, indeed, but says, "I just don't have the time to set tests and exams, and do the marking, so now I only do post-graduate supervision and exam moderation, as well as serving on the Academic Advisory Committees. I also present regular courses for SARF on unpaved roads, low-volume paved roads and stabilisation, and have also given courses in New Zealand and the USA."

INVOLVEMENT IN PROFESSIONAL BODIES

As a committee member of the SAICE Geotechnical Division and a Council member of SAIEG (South African Institute for Engineering and Environmental Geologists) over many years, Phil has tried to foster closer relationships between the two organisations, as they are undeniably linked. He believes that the two bodies are currently very close to each other and, as the need for CPD points for engineering geologists grows with the recent introduction of this requirement for natural scientists, mutual association will get even closer. SAIEG is also working with ECSA (thanks to SAICE's Peter Day for his on-going involvement) in the tricky area of job description.

ADVICE TO YOUNG ENGINEERS

Phil has mentored many young engineers, and his advice to them can be summarised as follows:

- Improve your qualifications.
- Learn from the people around you.
- Don't job-hop for the sake of money; rather build a steady career.
- Do what you do do well (from a '60s song by Ned Miller)

ADVICE TO ENGINEERS IN GENERAL

Nearing retirement age himself now, he cautions against specialising in too narrow a field, which could limit one's employment potential in later years quite considerably. Phil's expertise is in fact a good example of diversification, as, within his specialised field (construction materials) he is skilled in paved and unpaved low-volume roads, cement and lime road stabilisation, the geology of roads and the forensic aspects thereof, the management of potholes (identifying the cause before patching), and the evaluation of proprietary soil stabilisers.

FUTURE PLANS, PHILOSOPHY OF LIFE

Phil quips that some of his geological colleagues older than him say, "We never retire, we seem to weather with time and eventually decompose in the ultimate test pit." Although Phil hopes to start slowing down soon, he is still fully occupied, mostly on projects beyond our borders.

He would love to spend more time off the beaten track, watching and photographing birds. His family have in any case become used to repeated stops on holiday trips so that he can take photos of geology, roads and other engineering things!

Verelene de Koker
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Southern Cape Landslip, **Mossel Bay**

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This case study highlights one of the most significant landslips in the Southern Cape in recent South African history.

Figure 1: Aerial photo of the area affected by the landslip

BACKGROUND

Towards the end of 2015, attention started to be drawn to several residential properties in the suburb of Hartenbos in the coastal town of Mossel Bay in the Southern Cape (refer to Figures 1 and 2), which were showing signs of severe cracking and structural distress (refer to Figures 3–5). A preliminary geotechnical study was commissioned by the Home Owners Associations of the two affected complexes to assess the cause of the problems observed. The preliminary investigations indicated

Figure 2: Oblique aerial images of the affected residential developments

Figure 3: Tension crack observed in the ground next to the upper estate

that a deep-seated landslip was slowly developing between the two complexes.

Initial observations indicated that approximately 30 residential houses were affected by the landslip, and due to the high safety risk, residents were evacuated from zones showing significant vertical and lateral displacement of the ground. Subsequent to the initial investigations, Mossel Bay Municipality commissioned a more detailed study to further investigate the problem and assess possible solutions. To date, over 45 houses have been affected, several of which have been condemned and demolished. The scale of the problem is yet to be fully understood.

This case study highlights one of the most significant landslips in the Southern Cape in recent South African history, and it not only demonstrates the role and responsibility of civil engineers in our society, but also the potential scale of emotional distress caused to society when geotechnical uncertainty is not defined and interpreted. The findings discussed in this article also highlight the fact that geotechnical conditions which may impact residential developments (or any civil engineering project) may extend beyond the footprint of any particular structural element.

AIMS AND OBJECTIVES

The aim of the detailed investigations was to determine the trigger mechanism of the landslip and the depth of the failure zone, as well as the feasibility of possible solutions identified in the process. Due to the continual movement of the ground and the urgency of the matter, time was of the essence in the investigations.

PROJECT DESCRIPTION

The area affected by the landslip consists of two group residential complexes, which are separated by a steep embankment. The upper complex is situated in an old gravel quarry, and the lower complex in an old clay quarry. The affected area also extends into an adjacent private residential neighbourhood.

Site geology

The geology of the site was important in understanding the origin and mechanisms of this landslip. The landslip area is underlain by a thick sequence

Figure 4: Typical distress observed in many of the residential structures

of alluvial sediments of the Uitenhage Group, which consists locally of the younger Buffelskloof Formation and the older underlying Kirkwood Formation. The depositional environment was a dynamic coastal embayment created under fluctuating sea levels, with interfingering of marine, estuarine and alluvial sediments. The Kirkwood Formation was deposited in a low-energy fluvial environment, and consists mainly of mudstone and fine sandstone. Subsequent continental uplift and regression of sea levels resulted in increased erosion of the interior Cape Fold Belt mountains, and rivers flowing from these high-lying areas bisected lower-lying alluvial terraces, depositing thick sequences of sand,

Figure 5: Shearing of structures clearly indicated in several of the openings

Figure 6: Tension crack with 500 mm vertical displacement

Figure 7: Tension crack extending several metres into the ground with 200–300 mm horizontal displacement

gravel and cobbles (conglomerate) of the Buffelskloof Formation in gullies and large alluvial fans.

At the site under investigation, the Kirkwood Formation is exposed on the lower part of the site, and the Buffelskloof Conglomerate is exposed on the upper part. The contact between the two formations is exposed along the steep embankment between the two residential complexes. The contact between the two formations dips towards the base of the slope.

Site investigation methods

Several shallow test pits, as well as rotary core boreholes, were drilled across the site. Cross sections were then taken between the boreholes in order to establish a geological model of the site, and samples of residual Kirkwood clay were taken to determine indicative shear strength parameters in the laboratory.

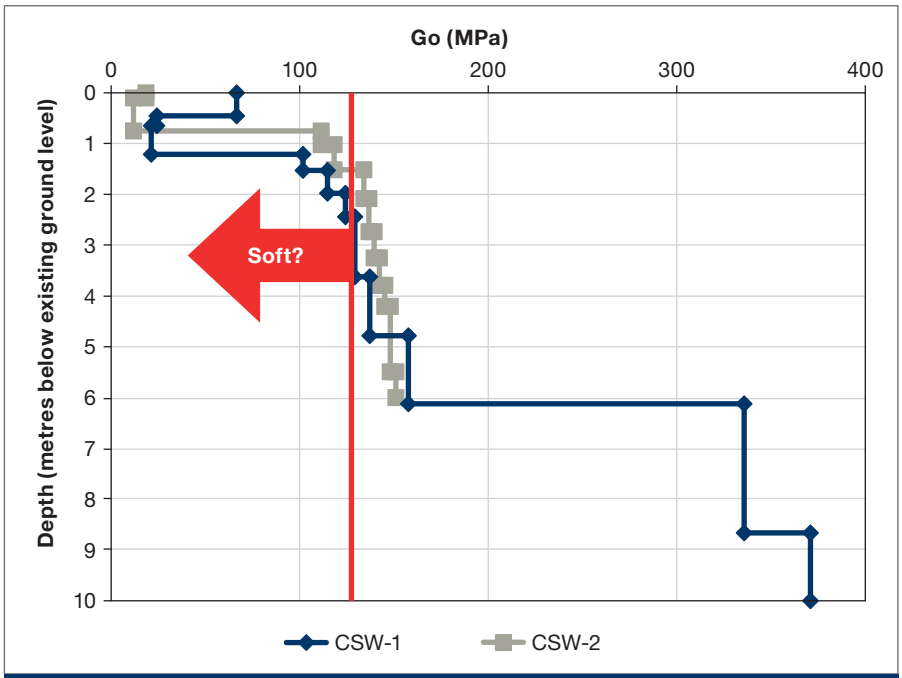


Figure 8: Continuous Surface Wave (CSW) test results

The striations (slickensides) observed at particle level demonstrate that the upper Kirkwood (composed of 88% to 92% clay/silt) has been sheared along distinct failure planes.

A high-precision three-dimensional (3D) survey was undertaken of several structures in the area to determine the magnitude and direction of movement. Due to time constraints, the installation of geotechnical instrumentation was not possible. Following the high-precision survey results, it became evident that such instrumentation (inclinometers, etc) would probably have been damaged in a relatively short period of time by the extent of the movement measured.

Continuous Surface Wave (CSW) tests were also employed to assess the ground stiffness profile and help identify zones of soft or weak ground. The CSW test positions were situated in locations where the perceived slip plane associated with the landslip was assumed to be close to the existing ground surface.

A 2D model of the slope was then generated using computer software, and a slope stability analysis was undertaken to assess conceptual failure mechanisms.

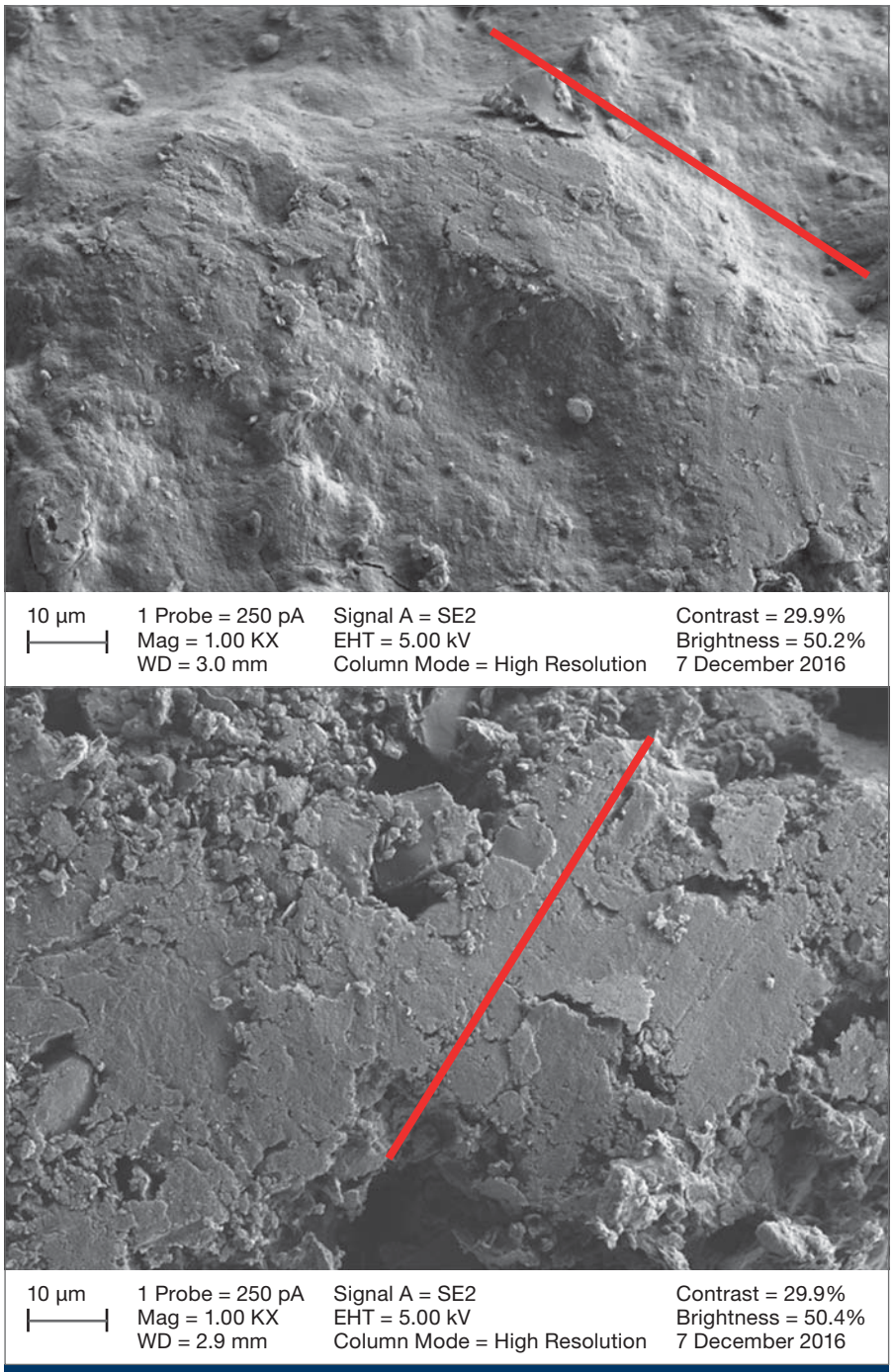


Figure 9: Microscopic view of soil fabric showing striated, slickensided orientation (general direction shown by red lines)

PROBLEMS ENCOUNTERED AND INNOVATIONS

It became evident during the core drilling operations that the ground was moving continuously, and it resulted in the contractor drilling through his lower steel casing on quite a few occasions due to the ground movement below. The boreholes essentially became crude inclinometers, information which was later used to model the slope stability and determine the depth of the problem. The extent of the ground movement is shown in Figures 6 and 7.

The presence of abundant gravel (pebbles) resulted in poor recovery during drilling operations and an inadequate assessment of the shear strength properties associated with the landslip. Good core recovery was, however, obtained in the Kirkwood clays, and one of the failure planes was recovered in the core. This allowed for more accurate sampling for shear strength tests.

RESULTS OF THE INVESTIGATIONS

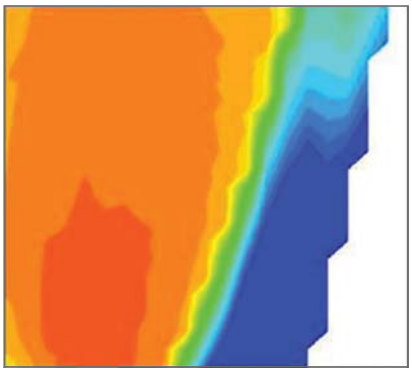
The geological model that was constructed from the drilling data indicates a sloping palaeo-channel in the Kirkwood clay, which is now filled with Buffelskloof Formation conglomerate. Seepage of groundwater was also noted where the palaeo-channel daylights on the sloping embankment between the upper and lower residential developments.

The CSW test results, presented in Figure 8, clearly indicate that the upper Kirkwood Formation was of a very low stiffness (soft consistency).

The upper Kirkwood Formation was identified as the zone in which the landslip is occurring. Consolidated, undrained shear strength test results demonstrated very low cohesion and friction angles that varied between 10° and 21°. Some interesting information and discussions were noted on several occasions by experienced laboratory testing staff who performed the shear strength testing. Further microscopic assessment was

Model 1: Existing conditions

Material name	Color	Unit weight (kN/m³)	Strength type	Cohesion (kPa)	Phi (deg)	Water surface	Hu type	Ru
Reworked Buffelskloof Formation		18	Mohr-Coulomb	0	32	Water surface	Constant	
Upper Kirkwood		16	Mohr-Coulomb	2	10	Water surface	Constant	
Lower Kirkwood		19	Mohr-Coulomb	2	32	None		0



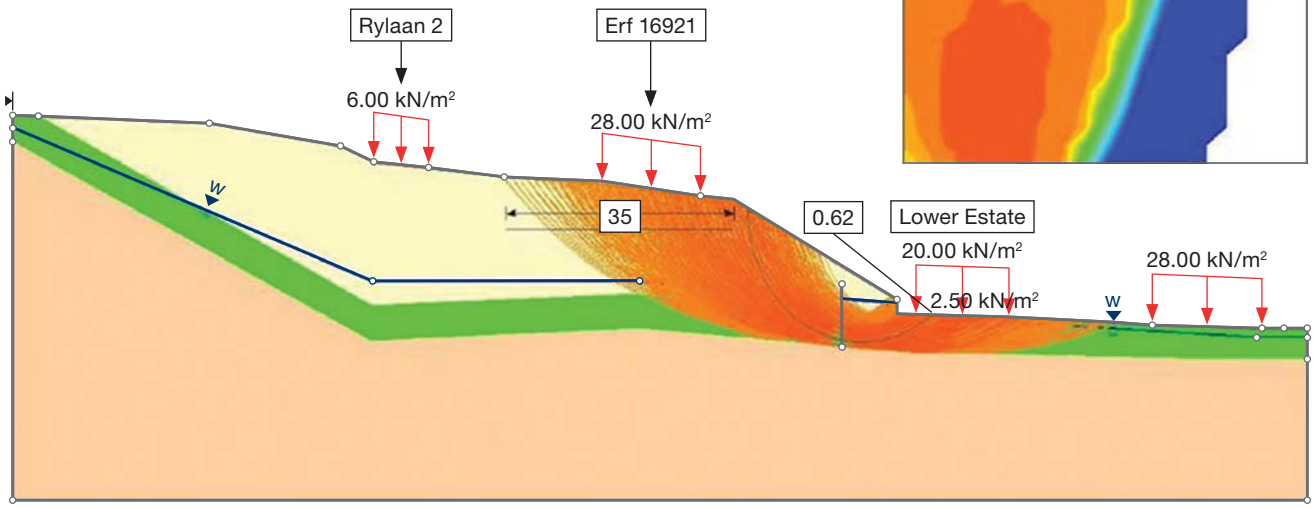


Figure 10: Conceptual 2D slope model showing failure plane analysis

performed by SCI-BA Laboratories and Scientific Consultants on undisturbed upper Kirkwood material, and a few interesting characteristics were observed (see Figure 9).

The striations (slickensides) observed at particle level demonstrate that the upper Kirkwood (composed of 88% to 92% clay/silt) has been sheared along distinct failure planes. Depending on the degree of surface water ingress and penetration into the Kirkwood, the disturbed mass is expected to continue moving. Furthermore, it is interesting to note that the landslip movement is tending to reflect the monthly rainfall patterns in the Mossel Bay area.

Analysis of potential failure planes, using 2D slope modelling, indicates very low factors of safety along semi-circular planes in the upper 10 m of the Kirkwood clay. Potential failure planes extend from the upper complex/estate, where tension cracks are visible, to the lower comlex/estate, where a classical bulging toe of the embankment is observed. The conceptual slope model is shown in Figure 10.

CONCLUSION

This investigation has established that a significant slope failure is occurring in the upper part of the Kirkwood Formation which underlies two built-up residential estates in Mossel Bay, resulting in severe structural distress to approximately 45 houses. Significant damage to municipal services has also been reported. The cost of this natural disaster is estimated to be in the millions of rands, and has had a major social impact on the area. The ground movement is on-going and the affected area has not been firmly delineated. At the time this article was written, a conceptual solution has been proposed and is being considered by the authorities. Certainly, the cost of any solution will be extremely expensive, and unless sufficient funding can be acquired, the future of the area remains uncertain.

Questions that need to be asked include whether this disaster could have been predicted – a question that does not have a simple answer. Certain elements of the problem, such as poor soil conditions, groundwater seepage and steep slopes, definitely point towards a potential slope failure, albeit with 20/20 hindsight vision, but preventing development under these conditions would preclude large parts of the Southern Cape.

This case study should, however, serve as a reminder of potential geotechnical problems that could be encountered in future development of this area. As has been noted in previous civil engineering articles, geotechnical investigations are often overlooked or tend to be heavily constrained by time and costs, rather than ensuring a thorough understanding of the ground conditions and potential risks that may impact a development. The local and national civil engineering fraternity need to take into account the potential consequences of geotechnical uncertainty, as the social impact to the public we serve may be far-reaching, with disastrous long-term effects. ◻

PROJECT TEAM	
Client	Mossel Bay Municipality
Consultants	Kantey & Templer (Pty) Ltd
	Outeniqua Geotechnical Services
Geotechnical drilling contractor	Geopractica Contracting (Pty) Ltd
Project value	Unlimited – people's lives have been changed forever

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St Helena Airport dry gut rockfill

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INTRODUCTION

St Helena is one of the most geographically isolated islands in the world, located approximately 1 950 km from the southwest coast of Africa. Since the island's discovery in 1502, the only access has been by sea, with the maximum size and weight of any single component being determined by the fact that it had to be transported by the mail ship RMS St Helena.

The economic viability of St Helena is dependent on the frequency and reliability of access for people and goods to the island. The airport project was destined to change the lives of all Saints, not only in providing employment with opportunities of developing new skills, but ultimately boosting the island's economic growth with increased tourism, and stimulating the expansion of support industries.

Terraced dry gut rockfill – total height 102 m

Start of 8 million m³ dry gut rockfill

Dry gut fill – preparing platform for initial trial embankment

The airport project provided many unique and unusual features requiring advanced engineering ingenuity and planning. The remote location of the island necessitated major logistical considerations, as almost everything, excluding rock and water, had to be shipped to the island. The island's heritage and fragile environment – a significant legacy of international acclaim – also necessitated careful consideration and detailed design and construction planning.

AIRPORT INFRASTRUCTURE – DRY GUT ROCKFILL

The construction of the runway entailed the following:

- Bulk earthworks for the airfield, which required drilling and blasting of predominantly basaltic igneous rock for the dry gut bulk fill located at the southern end of the runway (with 100 m high terraced profile requiring 8 million m³). This aspect of the project presented the biggest risk in ensuring stringent final level tolerances (6 mm in 3 m straight edge) to support the concrete runway.
- Sourcing of sufficient water for processing the rockfill, and construction of temporary storage dams (4 × 2 million litre HDPE-lined facilities).
- Considering the construction of a dam to attenuate runoff from the dry gut catchment to facilitate controlled upstream flow through a 3 m × 680 m long box culvert. Further value engineering resulted in this idea being discarded in favour of the excavation of an open channel drain, and the material being used for balancing the dry gut fill requirements.
- A 2 km long concrete surfaced runway giving an effective 1 550 m available landing distance.

The strength and settlement characteristics of the available materials to be used in the rockfill determined the side slopes and construction processing performance of the fill embankment.

The total fill required for the construction of the runway platform was sourced within the airport development area. The terrain characteristics of the dry gut presented a steeply-sided valley at the southern end of the runway. The fill extends beneath the runway end safety area where post-construction settlement needed to be minimised.

Design criteria

The design criteria for the runway strip were specified as follows:

- Earthworks to comply with OTAR Part 139
- Allowable tolerance for concrete runway surface 6 mm in 3 m straight edge
- Design life of 120 years for the earthworks structure.

In rockfill structures the aim is to compact the material to form a dense matrix and maximise settlement during compaction, as well as the interlock between large hard rock particles. In line with rockfill dam construction methodology, it was proposed to use a construction method-based specification. This method was refined following the results of field tests during the construction process, and following extensive trials on site, particularly during the early stages of the fill construction.

Site investigations

The survey control system used was based on the St Helena local coordinate system and consisted of 18 beacons covering the footprint of the airport site and dry gut, and was set up using post-process static, post-processed kinematic GPS data (PPK), and GPS survey methods.

Geotechnical investigations were conducted on Prosperous Plain and the dry gut for the mapping of exploratory drilling, trial pits, borehole cores, discontinuity surveys and the analysis of the borehole core logs and laboratory data. It was concluded that, on average, approximately 60% of the excavated material was very hard, with a UCS of 100 MPa, and would not break down during processing of the fill materials. This material consisted mainly of trachyandesite (average specific gravity of 2.6). The other 40% of the excavated material consisted of soft or decomposed rock with a UCS of less than 100 MPa, which would break down during processing.

During the initial stages of the contract extensive groundwater investigations were conducted, with 18 boreholes drilled in the vicinity of the airfield and surrounding areas. Pump tests indicated positive groundwater yields for construction water requirements (1.6 million litres per day).

Due to the limited number of known (recorded) seismic events in the area, it was not possible to perform the usual probabilistic analysis, and consequently a deterministic seismic hazard analysis was conducted for the project requirements. The outcome of this analysis indicated that the island has a low seismic hazard potential, with a predicted mean peak ground acceleration (PGA) of 0.021 g and an upper limit (maximum) PGA of 0.05 g.

Materials performance and method of construction

Prior to bulk placing of rockfill, an initial trial embankment (ITE) test section was constructed, using different combinations of material types, sourced from the cut zones. The water quantity that was added was varied, and different compaction efforts with the equipment available were tested to verify the best placing methodology.

The test results indicated the most effective construction application for the material sourced to achieve the target density of at least 80% relative density, i.e. optimally processing an 800 mm thick layer compacted by 10 roller passes using a 20 tonne smooth drum-vibrating roller after 80 litre/m³ of water had been added.

Laboratory sample after grading

Two embankment zones were constructed, namely an inner zone directly under the runway and an outer zone forming the outer embankment slopes. A third drainage zone was constructed between the natural valley slopes and the fill, and along the bottom of the dry gut valley. The minimum fill layer thickness was controlled with the maximum particle size not exceeding two-thirds of the layer thickness.

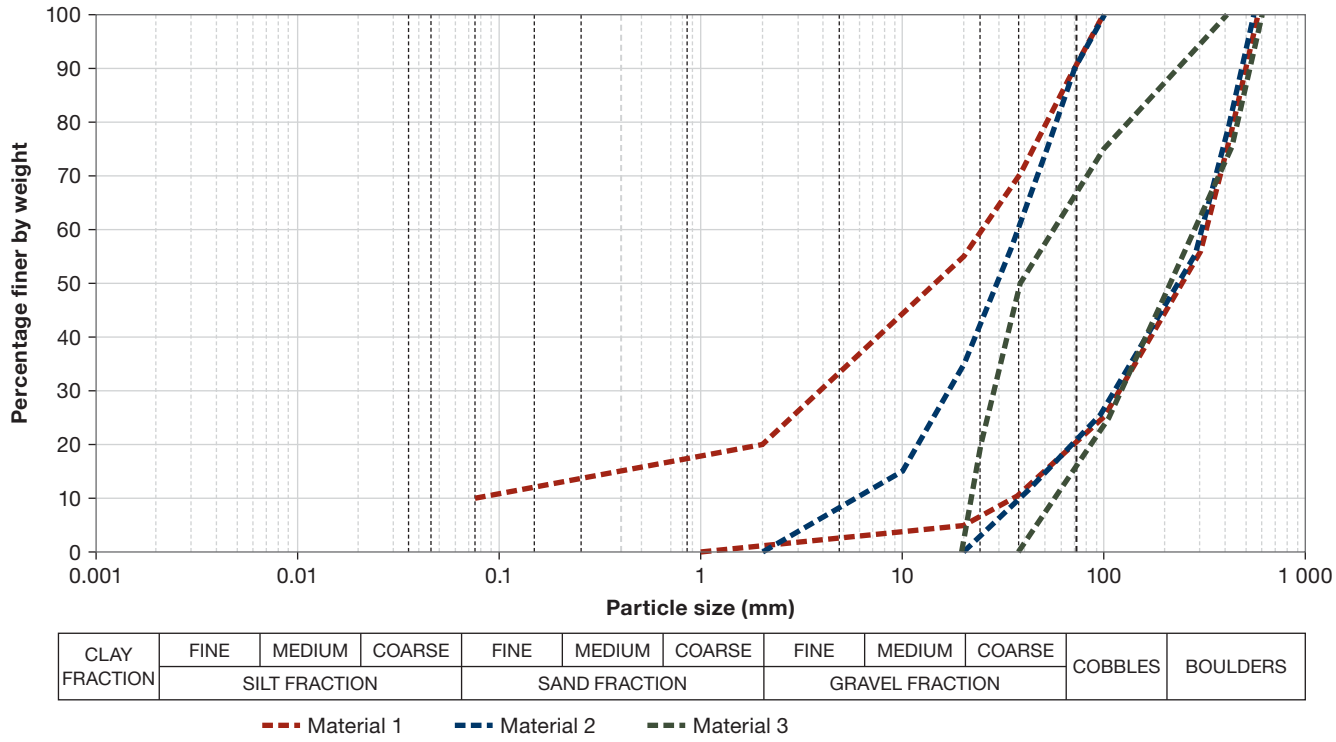
Mixing and blending of the rock material was achieved during the normal excavation and placing process. The composition of the source material varied significantly and was tested in the field at various source locations. A ratio of 60% harder trachyandesite rock to 40% softer basalt rock was considered desirable.

Placing took place over a wide front to facilitate a high production rate – approximately 15 000 m^3 to 20 000 m^3 per day utilising a double-shift 24-hour-day production strategy.

Grading of rockfill materials

Considering the materials available, it was recommended that only three different categories of material be used for the construction of the fill. The target grading for all materials was characterised as "well graded" with respect to gravel content – where the coefficient of uniformity (Cu) was exceeding 4, and the coefficient of curvature (Cz) between 1 and 3 for a well graded material. It was decided that "poorly graded" material would also be acceptable, as long as the general grading was within the specified ranges.

- ***Material 1*** formed the bulk of the rockfill embankment and comprised a blend of the uncontrolled blasted hard rock and the softer rock materials ("well graded" material).
- ***Material 2*** was used on the exposed embankment slopes for added slope stability and protection against the elements, and consisted of hard rock material only ("well graded" material). Adequate volumes of this type of material was stockpiled separately to ensure the required volume would be available for the construction of the outer section of the benched slopes. Material 2 was placed in a 4 m to 5 m wide zone on the outer surface of the benched slopes, from the bottom to the top of the rockfill embankment. The target grading of Material 2 was such that 15% of particle sizes less than 19 mm were excluded and point-to-point contact was retained for particle sizes greater than 50 mm.
- ***Material 3*** was used as the drainage layer in the base of the dry gut channel and as a drainage interface layer to continue up the valley sides to provide the lowest resistance against flow,



SAINT HELENA AIRPORT GEOTECHNICAL INVESTIGATION

SPECIFIED GRADING LIMITS

SCALE N.T.S.

Figure 1: Graphical display of the grading limits for the rockfill material

should water enter the fill ("poorly graded" material).

A graphical display of the grading envelopes for the rockfill material is shown in Figure 1.

Rockfill embankment slope stability

Detailed analyses were performed using the US Corps of Engineers' approach for dams, which defines 1.5 as an allowable FOS for embankment slope analysis.

Without the availability of conclusive laboratory tests at the initial stages of the project to guide the selection of shear strength parameters to use in the slope stability analysis, empirical methods were used to derive acceptable shear strength parameters. Research has shown that the following relationship can be used to determine the shear strength parameter ϕ (with c = 0 kPa) for rockfill, gravel and sand:

$$\phi = \phi_0 - \Delta\phi \log\left(\frac{\sigma_3}{1 \text{ atmosphere}}\right)$$

Where:

ϕ = the friction angle

ϕ_0 = the friction angle at 1 atmosphere pressure (101.3 kPa)

$\Delta\phi$ = the correction for confining pressure variation

σ_3 = confining pressure in the fill

From several laboratory test results on materials from various dams the typical ϕ_0 and $\Delta\phi$ ranges can be determined as summarised in Table 1.

Table 1: Typical ϕ_0 and $\Delta\phi$ material ranges based on laboratory tests

Material	Relative density (%)	ϕ_0 (°)	$\Delta\phi$ (°)
Rockfill	100 50	55 45	10 8
Gravel	100 50	51 41	8 3

In the case of high internal confining pressures (say fill heights > 50 m) it can be shown that ϕ may vary between 41° and 45° for rockfill at between 50% and 100% relative density, while the corresponding values for gravel may vary between 40° and 43°. Normally the compaction of rockfills should be at least between 75% and 80% of the bulk relative density. The composition of the rockfill that was used to construct the dry gut embankment contained a substantial gravel content, hence ϕ = 42° was proposed as an acceptable shear strength parameter for deeper-seated failures. A bulk rockfill density of 2 150 kg/m^3 was used.

However, where c = 0 kPa in slope stability analysis, shallow slope failures with low factors of safety are the most critical, as was the case for the bench slopes selected. In this case the confining pressures were much lower, and for the rockfill ϕ varied between 47° and 52° (44° and 48° for gravel). It was therefore proposed that ϕ = 46° to ϕ = 49° should be used in the analysis, which was consistent with the values and ranges as recorded in Table 1.

The finally proposed rockfill embankment slope geometry was as follows:

- Benches 10 m high at a slope of 1:1.36 with a 4 m wide horizontal surface between benches, which equates to an average (relative) slope of 1:1.76.
- Maximum embankment height 110 m.

The shear strength parameters of the excavated material were re-tested once exposed in the initial stages of the excavations to verify the above-mentioned analysis. The following tests and observations were conducted on the initial trial embankment procedure (and were repeated for any change in material composition) to verify the material characteristics and performance (initially

at a frequency of approximately 50 000 m³ intervals, and once a level of consistency was achieved, at every 100 000 m³ during the construction progression):

- Plate load tests
- Large volume density tests
- Grading analysis
- Water absorption and porosity
- Wash-out trial tests to monitor the optimum water demand required to achieve interlocking of the rock fragments
- Compaction effort against settlement measurements.

This data, together with numerical analysis, provided confidence in the construction methods used and in determining the final construction levels to accommodate the projected settlement, and hence ensuring that the upper surface of the fill remain within the prescribed tolerances throughout its design life.

Settlement

It is very difficult to quantitatively predict the settlement of a rockfill embankment, and therefore the experience gained at different rockfill dams in southern Africa was considered, together with international experience documented on concrete-faced rockfill dams.

The embankment settlement was regularly monitored and assessed with installed settlement monitoring equipment and geodetic survey measurements during the construction phase. This data, together with numerical analysis, provided confidence in the construction methods used and in determining the final construction levels to accommodate the projected settlement, and hence ensuring that the upper surface of the fill remain within the prescribed tolerances throughout its design life.

Stormwater drainage

Due to the difficulty of constructing drainage collection channels at the interface of the toe of the fill with the natural rock valley sides, stormwater runoff from the airfield footprint was directed away from the fill matrix and channelled via outlets to convenient locations along natural contours and into the neighbouring water courses.

Also of note was that an open channel drain was excavated into the southern face of the dry gut (1 million m³) to redirect the stormwater flows from the upper dry gut catchment into the neighbouring valley, thus resulting in the omission of the initially proposed dry gut culvert and attenuation dam.

Balance of earthworks, design drawings and volume calculations

The surfaces of the embankment profiles and volume calculations were generated from *Model Maker TOT* files of the site survey data. Due to the initial uncertainty of the ultimate performance of the

A haul road of 14.5 km had to be constructed over very harsh, rocky and steep terrain

Fixing the final vertical alignment of the runway was vital, considering the lead time required in setting up the procedures for the necessary flight path sensitivity analysis and the follow-on requirements with early submissions to the Aviation Regulator for final approvals.

source materials, a sensitivity analysis was computed.

Fixing the final vertical alignment of the runway was vital, considering the lead time required in setting up the procedures for the necessary flight path sensitivity analysis and the follow-on requirements with early submissions to the Aviation Regulator for final approvals. As such any later adjustment to the runway alignment would have had serious time-delay consequences for the construction programme.

The earthworks volume-sensitivity analysis clearly showed that the slightest variation in the material performance would have a marked effect on the material balance.

CONSTRUCTION CHALLENGES

The biggest challenge in constructing the St Helena Airport was creating and maintaining an efficient planning and logistics chain. There were no major construction plant or building materials on the island, and virtually everything had to be shipped to the island. The contractor chartered a 2 500 ton ocean-going vessel for the duration of the contract to accommodate their plant and materials supply requirements. With no harbour on the island, a temporary jetty had to be constructed to accommodate roll-on-roll-off facilities. Other early works establishments consisted of a temporary fuel facility (1.5 million litres), construction of a 14.5 km haul road over very harsh rocky and steep terrain, borehole explorations to source adequate groundwater for construction water, construction of staff accommodation and workshops, the establishment of a fully equipped internationally accredited laboratory, and the erection of crushing and concrete batching plants.

Risk awareness during project execution was absolutely crucial for the success of the project, and both the St Helena government and the project team ran and shared a comprehensive risk and opportunity register. This was vital to identify and mitigate any risks to the health and safety of personnel, and to protect the special features of the island prior to the commencement of any sector of the works. This approach was instrumental to the ultimate goal of successfully completing the contract on time and within budget (construction works for the airport infrastructure was completed in April 2016, and an Aerodrome Certification was issued by ASSI on 10 May 2016). ◻

PROJECT TEAM	
Client	Department of International Development, UK Government
Main Contractor	Basil Read (Pty) Ltd
Contract	Design Build Operate

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Photo 1: The presence of a waterfall is typically the most prominent indication that there are possibilities for small hydropower development

Some geotechnical aspects of small hydropower projects in Southern, Central and East Africa

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INTRODUCTION

Increasing energy demands and increasing demands for greener energy – particularly in the developing world – have seen a growing focus on renewables. Small hydropower (i.e. up to 20 MW) is an important part of that mix. While most of Southern Africa is water scarce and therefore has limited potential for small hydropower generation, many possibilities exist in West, Central and East Africa. Geotechnical factors are major considerations in many aspects of implementing such projects, including evaluating the viability of various options, and considering the design, construction, operational and maintenance impacts.

TYPICAL PROJECT ELEMENTS AND SITE CHARACTERISTICS

Hydropower generation is not in itself a new concept, and in essence depends on two main parameters – the water flow and the available head. Even relatively small rivers with sufficient elevation difference might represent a viable small hydropower project. Larger rivers can get away with less head, and still represent feasible schemes.

Typical project elements include a diversion structure or dam within the river at the highest practical elevation, a desilting structure, the conveyance (likely

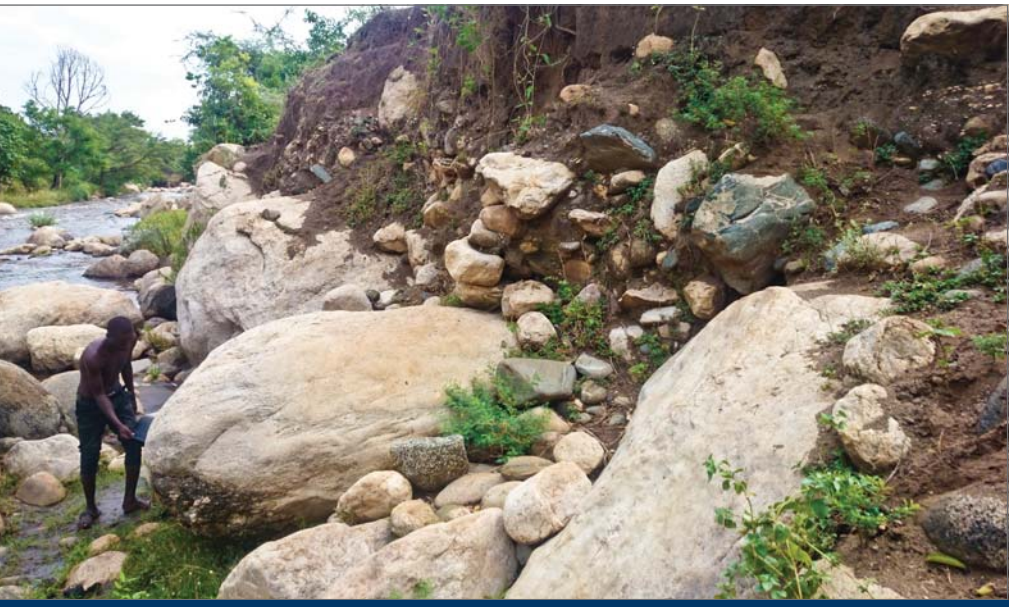

Photo 2: Thick boulder accumulations are likely to have been deposited during floods associated with ice sheet retreat at higher elevations, and are not a feature of current climatic conditions

Photo 3: The major slope failure in the background also destroyed the headrace canal – and shut down the power plant

Photo 4: A further example of very large boulders that predate the current flood regime, whereas the smaller boulders comprise the mobile bedload

comprising the headrace canal or pipeline), a surge tank or forebay, penstock and powerhouse. Once through the turbines, the water is returned to the river.

By virtue of the need for perennial rivers, many of the hydropower possibilities are located in the tropics band incorporating West, Central and East Africa, or in mountain ranges that might be associated with high rainfall. It follows that the topography is generally steep, and waterfalls are often a reflection of the sudden drop in elevation (Photo 1).

There are exceptions to the above, for example in the case of releases from the Lesotho Highlands Water Project, which have created hydropower generating possibilities in South Africa, even though the natural elevation differences are not large.

GENERAL REGIONAL GEOLOGICAL AND GEOMORPHOLOGICAL CHARACTERISTICS

In the tropics it is to be expected that the typical tropical soils and laterites might be developed, while deep weathering of the bedrock is also characteristic.

Parts of East Africa are further characterised by elevated levels of seismic risk due to the presence of the East African Rift System (EARS). Many of the small hydropower possibilities within this region are located either within or in close proximity to the Rift System. Such elevated seismic hazards need to be considered in designs.

In addition to the seismicity, many of these schemes are further located in areas of relatively young volcanic activity (Mount Kenya, Mount Elgon) where the geological succession is characterised by an alternating sequence of lavas (possible hard rock), weaker tuffs and other agglomerates.

Some of the mountains, such as the Rwenzori Mountains, comprise uplifted horst / fault blocks, and granite gneiss predominates.

In terms of the geomorphology, conditions encountered are not entirely shaped by current climatic conditions. For example, the thick alluvial deposits, including massive boulders beyond the capabilities of the current river regime (Photo 2), as encountered in many of the river valleys, have their actual origin in the various repetitive cycles of glaciation and retreat of the ice sheets. These ice sheets are still visible in places today (Rwenzori Mountains and also Mount

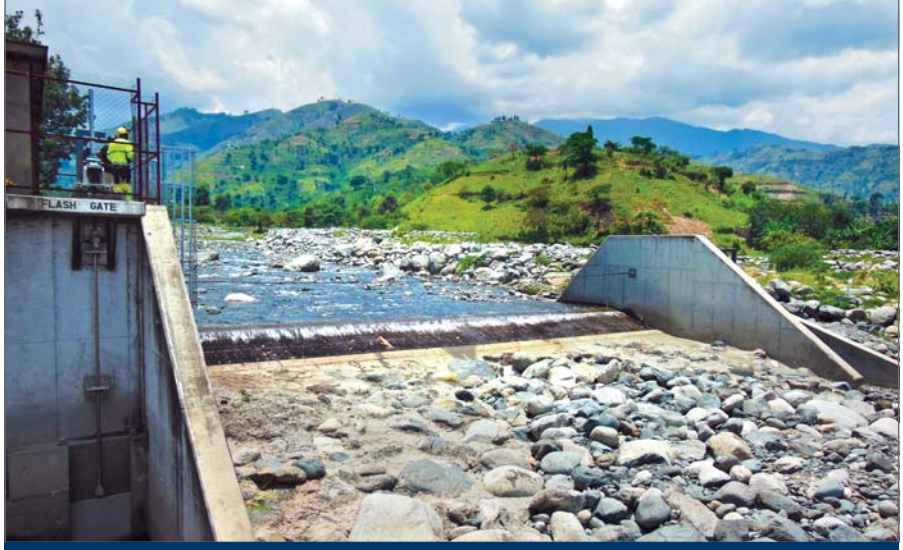

Photo 5: An example of a Tyrolian weir which allows the mobile bedload to pass through while the water is diverted

Kenya, for example), although they are much reduced. These past influences must be recognised in assessing the current geomorphological environment.

GEOTECHNICAL CONSIDERATIONS

Steep slopes

The typical steep slopes that are encountered, together with the characteristic deep weathering and thick soil cover in places, further impacted by intensive deforestation, create real issues of slope instability. The implications of such instability can vary widely – from a minor maintenance issue relating to the clearing of slipped material, to major failures that could result in loss of infrastructure, and also the power generation capability (Photo 3).

A further aspect is palaeo-instability, where elements of the scheme such as the diversion weir or dam, might be located on former landslide material. The effects of wetting-up following impounding might result in reactivation of the palaeo-slide, and should therefore be recognised at an early stage as a potential fatal flaw when evaluating the feasibility of such a scheme.

Founding conditions

Not all the components of such a scheme have high requirements for founding. The main elements where founding is a consideration include the weir (or dam), as well as desilting works, the forebay and the powerhouse.

For the diversion structure the presence of shallow bedrock cannot always be assumed to be present. In the case of mountains that were formerly covered with ice sheets, e.g. the Rwenzoris, the rivers sourced from these peaks are often choked with major boulder deposits of glacio-fluvial origin. Some of these boulders may have diameters in excess of 5 m and would not be mobile in current flood cycles, but represent flooding linked to earlier periods of ice sheet retreat (Photo 4).

In order to avoid the problem of a conventional weir structure filling very quickly with such a mobile bedload, the design of a Tyrolian weir is often adopted, where the mobile bedload effectively continues downstream unhindered while the water is diverted into the intake (Photo 5).

CONSTRUCTION MATERIALS

Although the concrete volumes are small on such projects, finding a suitable source of coarse aggregate is often problematic in areas characterised by deep weathering. Where extensive deposits of hard rock boulders such as granite or lava occur, the small-scale crushing of these boulders represents a viable solution. Commercial sources are typically a rarity in the more remote areas and generally are not a realistic alternative.

CONCLUSION

This article serves only to outline some of the geotechnical aspects that might impact on small hydropower projects. This is certainly not a comprehensive discussion on the topic. No single recipe exists, however, and all sites are unique. It remains crucial that the geotechnical practitioner must be able to 'read' the unique site conditions and consider the practical considerations. ◻

Town Hill pipe jack

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INTRODUCTION

In March 2015 a sink hole, approximately 4 m x 3 m wide and 3 m deep, developed in the fast lane of the N3 southbound carriageway on Town Hill between Pietermaritzburg and Hilton. Fortunately no motorists were injured. Immediately after the sink hole appeared, the lane was closed and the cavity was backfilled with rock underlain with bidim fabric. The lane was soon reopened after the surface had been patched with an asphalt seal.

Drennan Maud (Pty) Ltd was approached shortly thereafter to investigate the cause of the sink hole and provide remedial recommendations to prevent the likelihood of such future catastrophes.

It was soon discovered that the hole had developed directly above an existing 900 mm diameter Armco pipe that lay approximately 10 m below road surface level. The Armco pipe had been installed in the 1960s when the N2 was first constructed and later extended, with the widening of the N3 at Town Hill.

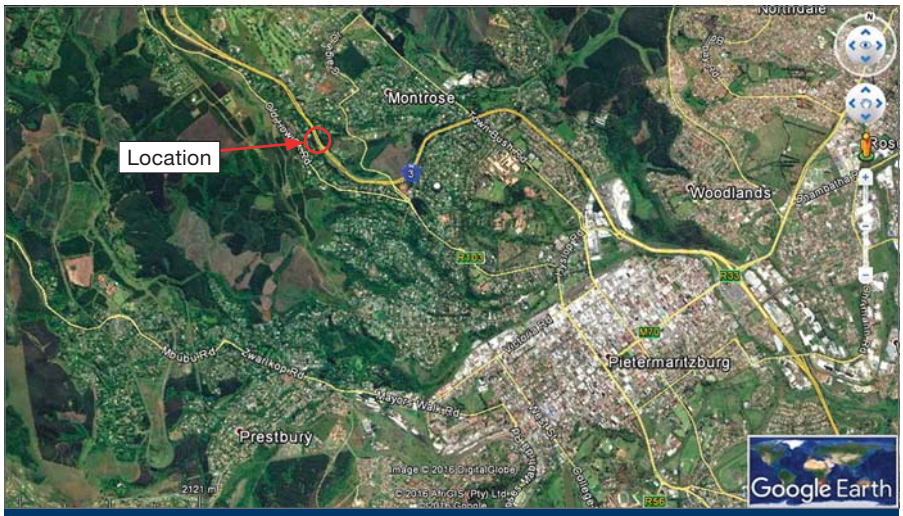

Figure 1: Town Hill pipe jack site location

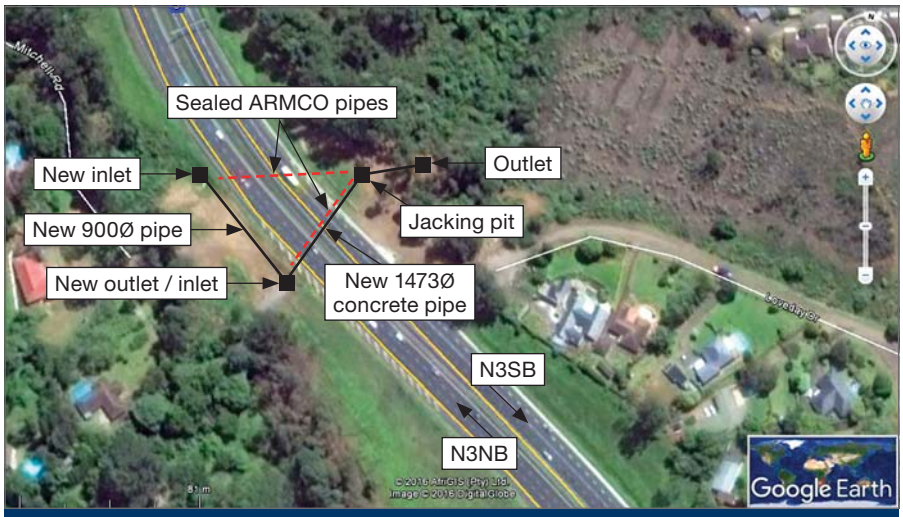

Figure 2: Existing and new drainage system

SITE DESCRIPTION

The immediate section of the N3 where the sink hole occurred is located on a fill embankment constructed on the lower slopes of the Townhill escarpment approximately 1 km north of the Peter Brown Bridge. Two drainage lines off the elevated north-eastern escarpment flank converge slightly downslope of this road section and were piped through the fill embankment when it was constructed.

The outlet from the pipes flow into the Town Bush stream, a tributary of the Msunduze River. Continuous seepage, likely to be gravity-fed by springs from the upper Townhill slopes, was evident down the drainage line, which increased substantially during the wetter summer months.

GEOLOGY AND SOILS

The area is underlain by the Pietermaritzburg Formation, which broadly comprises massive to laminated, deeply weathered, carbonaceous siltstone

Figure 3: Existing Armco pipe collapsed and damaged

and shale. A thick mantle of massive unsorted slump and talus material overlies the bedrock in this area. The slump comprises silts and clays with variable hard rock dolerite core stones (0.5 m to 2.0 m) in the finer material matrix. The material used in the construction of the fill embankment comprised predominantly slump and talus taken from the N3 cuttings further up the road. According to SANRAL (South African National Roads Agency Limited), some 'foreign' material could also be expected in the valley lines.

Three auger holes, excavated on either side of the N3 motorway and one in the median, revealed soft to moderately stiff, reddish brown silty clay or clayey silt. Fragments of deeply weathered, very soft (broke in the hand) shale were randomly found in the holes.

CAMERA SURVEY

Drawings provided by SANRAL suggested that two 900 mm Armco pipes originated approximately 75 m apart within the reserve of the northbound carriageway and converged to discharge across the N3 downslope of the southbound carriageway embankment.

As part of the investigation, a camera survey was commissioned. The pipes were surveyed with the aid of a remote-controlled camera to determine the condition of the pipes and the location of the junction box. The camera survey confirmed that the Armco pipe directly underneath the sink hole had completely collapsed. The survey further identified areas of corrosion, deformation and collapse along the pipe.

EMERGENCY WORKS

Given the poor condition of the Armco pipe, and to prevent the likelihood of further sinkhole development, it was decided to carry out temporary emergency works until the more permanent pipe jack solution could be implemented.

To stabilise the collapsed section of the pipe and cavities identified during the camera survey, probes were drilled around the pipe from the surface with an auger capable of drilling down through the stiff clayey horizon to a depth of 10 m, and simultaneously pressure grouting closed any voids that existed around the collapsed pipe. This was done to prevent the cavities from reflecting through to the surface and manifesting as sink holes.

The operation had mixed success in that the collapsed section of pipe beneath the sinkhole was sealed, but elsewhere

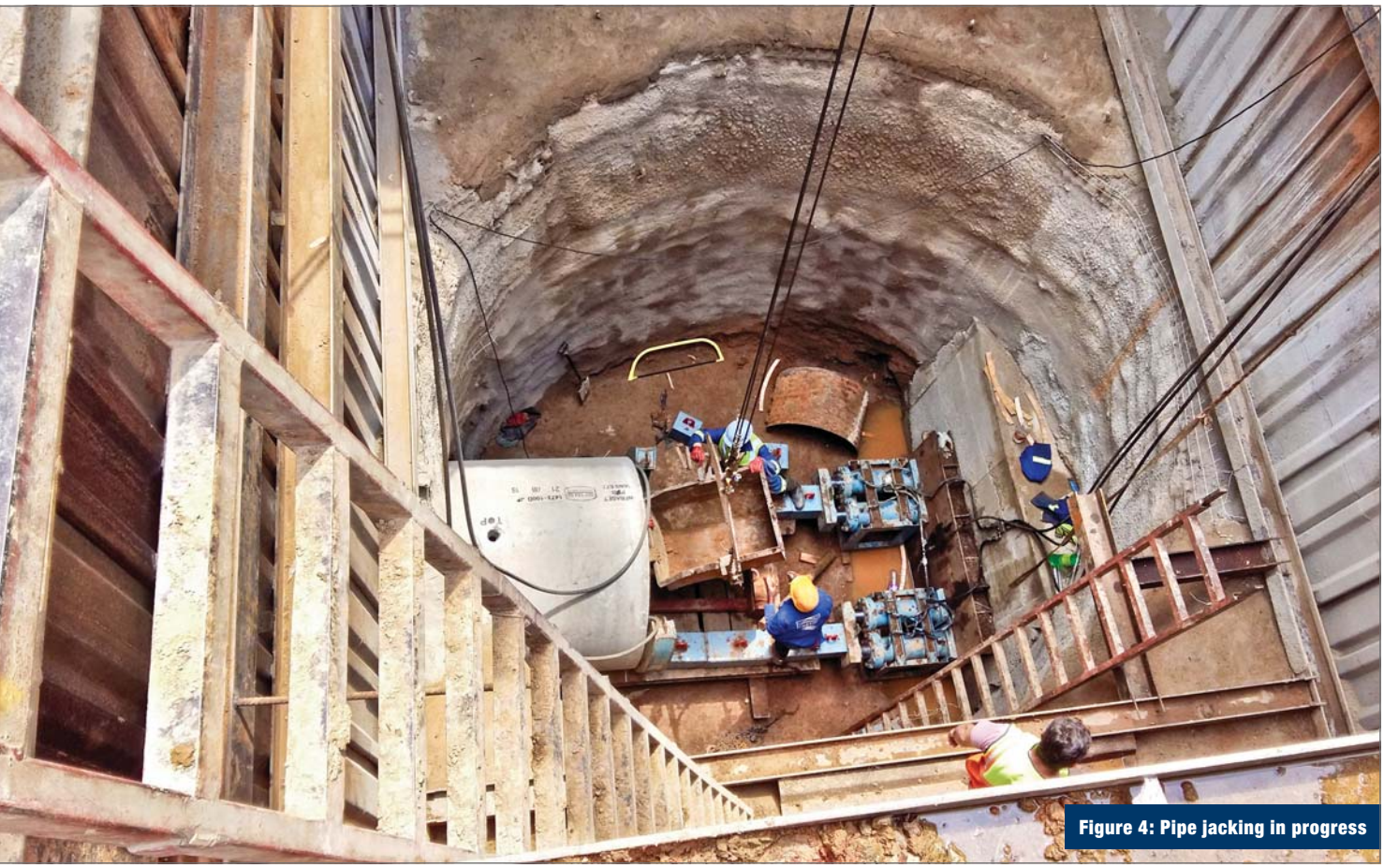

Figure 4: Pipe jacking in progress

Figure 5: New inlet for 900 mm concrete pipe

Figure 6: New inlet to jacked pipe with ancillary works

Figure 7: Side drain tying into new outlet (enlargement inset)

A number of the Armco pipes installed in the 1960s along this stretch of the N3 freeway have reached the end of their design life and will need replacing.

the poor condition of the pipe resulted in grout loss.

PERMANENT WORKS

The permanent works entailed jacking a 1 473 mm diameter concrete pipe, 70 m long, beneath the busy N3 freeway. The new pipe was jacked to encapsulate the existing Armco pipe, progressively enabling demolition and removal of the collapsed pipe. The greatest benefit of this approach was that it assisted in visually identifying voids and cavities along the Armco which were rectified by pumping in a weak grout bentonite mix to stabilise the voids before continuing. Larger cavities above the jacked pipe were sealed later by surface drilling and pressure grouting.

Simultaneously the inlet and outlet works were upgraded together with some ancillary drainage channels. Flow to the second Armco pipe located 75 m further upslope along the N3 was diverted to the new inlet before this pipe, too, was permanently sealed to prevent the possibility of similar sinkhole development in future.

Franki Africa, who carried out the work, completed the jacking operation in 20 weeks. A particular challenge was positioning the jacking pit correctly over the junction box of the two Armco pipes. The final depth of the jacking pit at 13.5 m required substantial lateral support measures. Given the steep gradient of the fill embankment slope, the outlet structure also required careful planning. Initially a series of cascades had been detailed to dissipate the flow energy, but was later revised to a piped section incorporating an energy dissipator founded on bedrock near the stream bottom.

Regular survey monitoring of the road surface showed minor vertical deformations during the jacking operations. Once the jacked pipe was through, further drilling and pressure grouting from the road surface were performed along the full length of the new pipe before laying the final 145 mm asphalt layer, at which time no further settlement was recorded.

CONCLUSION

A number of the Armco pipes installed in the 1960s along this stretch of the N3 freeway have reached the end of their design life and will need replacing. This project highlighted the effectiveness of trenchless pipe jacking, simultaneously allowing for removal of the defunct Armco pipe and replacing it with a new concrete pipe. The tunnelling operation meant that voids and cavities were assessed visually. Most of the remedial measures to seal cavities and voids were performed at depths ranging from 4 m to 10 m beneath the N3 freeway, thereby not interfering with the busy N3 flow of traffic. Where larger cavities were present, these were sealed from the surface during off-peak periods. ◻

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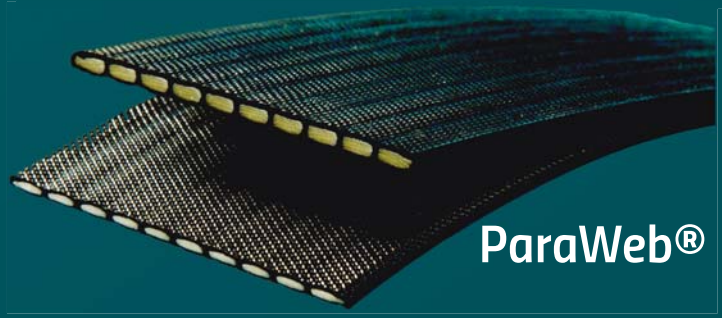

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Geotechnical works in progress on the Kranspoort Pass (November 2016)

Geotechnical engineering through the Kranspoort Pass

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INTRODUCTION

HHO Africa were appointed in 2011 by the South African National Roads Agency Limited (SANRAL) to carry out designs for the upgrade of National Route 11 Section 10 between Middelburg and Loskopdam, Mpumalanga. At the time, the steep 5 km Kranspoort Pass, which is located within this section of road, comprised a single descending lane and a single ascending lane, with limited climbing lanes. Sight distance and emergency stopping measures were also a concern, particularly with the increased use of the pass by heavy vehicles, and accidents were a regular occurrence on the pass. Safety improvements were therefore a key aspect of the project.

This article summarises the geotechnical investigations and design that were carried out at the Kranspoort Pass, and highlights some of the challenges that were revealed during construction and how these were addressed to ensure a safe journey through the pass.

PROJECT DESCRIPTION

The safety improvements to the Kranspoort Pass included widening to accommodate two lanes in each direction, geometric improvements, the installation of concrete barriers and the provision of two arrestor beds.

GEOTECHNICAL INVESTIGATIONS

Following initial visual inspections, which included limited joint measurements and assessment, a geotechnical investigation was conducted. This comprised drilling of rotary core boreholes along the proposed cut widenings. A total of twenty-one boreholes were drilled, with many of the setups proving to be very challenging on the steep terrain. The purpose of the

The completed gabion tie-back wall (December 2016)

Construction of soil nail tie-back gabion wall (February 2016)

boreholes, which consisted of both vertical and inclined drilling, was two-fold: (a) to sample the in-situ materials for pavement material classification, and (b) to establish the geology and geotechnical conditions for design of cut slope widenings.

The geotechnical investigations confirmed a composite geology through the pass, with a combination of jointed and variably weathered quartzitic sandstone rock, talus material, residual clays, as well as dolerite intrusions making up the complex geotechnical environment.

MATERIALS ASSESSMENT

Based on the findings of the laboratory testing of rock cores, material qualities could be attributed to the various proposed cut widenings, with recommendations for re-use as either fill, subbase, selected subgrade or base course material. Furthermore, assessment of excavatability could be made, with most of the cuts in quartzitic sandstone being classified as extremely hard ripping and requiring blasting.

GEOTECHNICAL DESIGN

Geotechnical design of the cut widenings essentially involved the installation

Construction works in progress on the main rock cut (November 2016)

of a trap and barrier, the width and height of each being determined by the nature of the cut material and design slope configuration. The initial design philosophy was generally to protect the road against falling and rolling rocks, rather than to attempt to stabilise the cuts. The exception to this being selected cut widenings that are discussed further below.

One of these cut widenings comprised highly weathered and rapidly decomposing dolerite, and consequently a soil nail tied-back gabion wall was designed at tender stage to address this.

CONSTRUCTION PHASE

Gabion tie-back wall

Immediately adjacent to the highly weathered dolerite that required the gabion tie-back wall was a section of quartzitic sandstone. During construction it was discovered that the design slope for this quartzitic sandstone section would result in the undermining of a large block of rock above the slope, and so it was decided that, rather than disturbing this block with the blasting and cutting back of the slope, the gabion tie-back wall would be extended across the quartzitic sandstone section.

Drilling and installation of the soil nails into the weathered dolerite went relatively smoothly, but the same cannot be said for the quartzitic sandstone section. The quartzitic sandstone was found to be locally very highly fractured and disturbed (presumably by the adjacent dolerite intrusion), resulting in blocks of rock falling into the drill holes and jamming the drill bits. Furthermore, the vibrations of the drilling were causing the jointed rock in the face surrounding the drill position to collapse and fall, making it unsafe for the drilling personnel.

A decision was made to install a flash covering of shotcrete to the highly fractured rock to facilitate the drilling and ensure safety of the drillers. The type of soil nail was also changed to a spin anchor, which requires a smaller diameter and shorter hole, and uses a fast-setting epoxy resin instead of grout. However, drilling and installation of these anchors still proved difficult, and their selection had to again be revised. Eventually Ischebeck TITAN self-drilling hollow bars were employed, and this achieved the required installation success within the fractured quartzitic sandstone.

It is of interest to note that a Chinese-manufactured alternative to the Ischebeck TITAN hollow bar was investigated, but upon scrutiny of the manufacturing specifications of this alternative, it was discovered that hot-dip galvanising of the bars would result in an unacceptable risk of hydrogen embrittlement, and therefore these bars were rejected.

PROJECT TEAM	
Client	SANRAL
Contractor	KPMM Construction
Geotechnical sub-contractor	Guncrete
Geotechnical suppliers	Geobrugg
	Ischebeck TITAN
	Maccaferri
	Reinforced Earth
Design and supervising engineer	HHO Africa
Construction value	R400m (Kranspoort Pass R40m)

Steeply-jointed rock cut

Upon widening of one section of cut, a set of roughly perpendicular and very steeply dipping joints was encountered. These joints were steeper than the design cut slope, but because the rock tended to break 'naturally' along these joints, achieving the design slope was impractical. The entire cut was constructed at the 'natural joint orientation', resulting in potential instability and sliding of rock columns that were formed by the perpendicular joints. Typically, the lower portions of the cut appeared to be more unstable than the upper portions, so a gabion wall at the base of the cut was proposed. With the original cut slope design having allowed for a trap at the toe of the slope, there was sufficient space for a Terramesh® wall to be constructed. The design of this structure was undertaken by Maccaferri, and comprised gabions with a four-metre extended tail. At the top of the wall, an additional row of gabions was added to act as a barrier for any loose rock that could possibly fall from the rock slope above and behind.

Main rock cut

The main rock cut on the Kranspoort Pass is up to 40 m in height, with a design slope of 65 degrees. Some 6 000 cubic metres of rock were due to come from the cut widening, with most of it being suitable for crushing into G1 base course material. Space constraints precluded the full required trap width from being constructed, and therefore additional protection was achieved by installation of a wire mesh. Geobrugg Deltax® G80/2 high-tensile steel was installed with rock dowels at 2 m horizontal and 2.5 m vertical spacing. The bottom cable was suspended 5 m above road level.

The major joint orientation in this cut had a strike roughly perpendicular to the cut face, which was not problematic. However, as the road made a bend

towards the end of this cut, the strike of this joint set became more parallel to the cut face, resulting in a significant toppling risk. This risk was greatest within the upper few metres of the slope, where the joints had naturally opened up over time and filled with transported soils. Blasting of this portion of the cut needed to be done carefully, and thus pre-splitting was recommended to minimise the disturbance and to not exacerbate the toppling stability concerns. The formation of a temporary route up to the top of the cut face by the contractor for access of blasting equipment formed a convenient bench that was incorporated into the cut slope design, with the wire mesh design above this bench modified to Deltax® G65/2, and with longer rock dowels to address the potential toppling within this section.

Main rock cut with wire mesh installation nearing completion (February 2017)

CONCLUDING REMARKS

The geotechnical works at the Kranspoort Pass are coming to an end, with the project completion due in August 2017. The complex geological setting and geomorphological environment of the pass demanded geotechnical designs that would be adaptable to the conditions. Good communication and interaction between the professional team, main contractor, geotechnical sub-contractor and geotechnical suppliers was vital in dealing with the various challenges encountered, and in successfully implementing the geotechnical designs on this project, some of which have been highlighted in this article. ◻

Wall 3 Versfeld Pass, **Piketberg**

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The recent upgrading of this pass, originally built by a local farmer with 16 labourers in just three months, has added a further chapter to its more than 70-year old history.

BACKGROUND

The Western Cape Government's Department of Transport and Public Works undertook the resurfacing and widening of the narrow curves on the Versfeld Pass to facilitate a safer and wider passage for the local agricultural industry and other users of the pass.

South Africa's mountain passes are always a civil engineering challenge, and this particular pass is one that comes with rich folklore, too, making it unique in its own right. There are in fact two Versfeld passes, but this article will focus on the existing one which was built in 1943, widened and tarred in 1958, and widened further now in 2016/17. The pass was originally built by a local farmer, John Versfeld, in three months using only 16 farm labourers and no dynamite. The pass is characterisd by its narrow and twisty nature, winding down the Piketberg Mountain.

OBJECTIVE

The aim of this study was to design a stable reinforced embankment (160 m long) to widen the existing road by approximately 7 m on a steep mountain side. Four walls along sharp curves were widened in the pass, and this article focuses on the geotechnical aspects of Wall 3.

GEOTECHNICAL CHALLENGES

A restricted field of vision along the narrow curved section hampered the ability to undertake the traditional methods of geotechnical investigation.

It was also evident in the field that the underlying geology would be highly variable, bringing to mind Karl Terzaghi's comment, "Natural soil is never uniform. Its properties change from point to point, while our knowledge of its properties is limited to those few spots at which the samples have been collected". This became very apparent once excavation commenced – a view of the underlying soils/bedrock revealed that the reinforced embankment would require design modification to incorporate Mother Nature's variances.

Figure 1: Aerial perspective of Versfeld Pass (Photo: Paul Fairbrother)

GEOTECHNICAL OPTIONS

The following geotechnical aspects were considered in the solution:

- The stability of the Reinforced Earth® system in terms of bearing stability on the side of the steep mountainside, especially in view of erosion-induced slope failures evident at the beginning of the project
- The need to maintain internal stability integrity so that settlement would not affect the road surface performance
- The application of traffic loading and the effects of stormwater runoff in the long-term consideration of global stability.

After careful consideration of the geotechnical aspects, and after regular consultation with the project team members on site, the reinforced embankment system showed in Figure 3 was proposed and installed. A major advantage of the system was the fact that modification of the various design elements to suit the localised rock variances could be undertaken with relative ease.

REINFORCED EARTH COMPONENTS

In sections where sufficient space existed behind the facing, a standard Reinforced Earth structure was used. In such cases the Reinforced Earth was used as the primary earth retaining system. A coherent gravity mass is created using steel soil reinforcements connected to a TerraTrel® facing which is made up of galvanised metal grids with rock packed behind.

In sections where a standard Reinforced Earth structure could not be

Figure 2: Existing site conditions on the Versfeld Pass

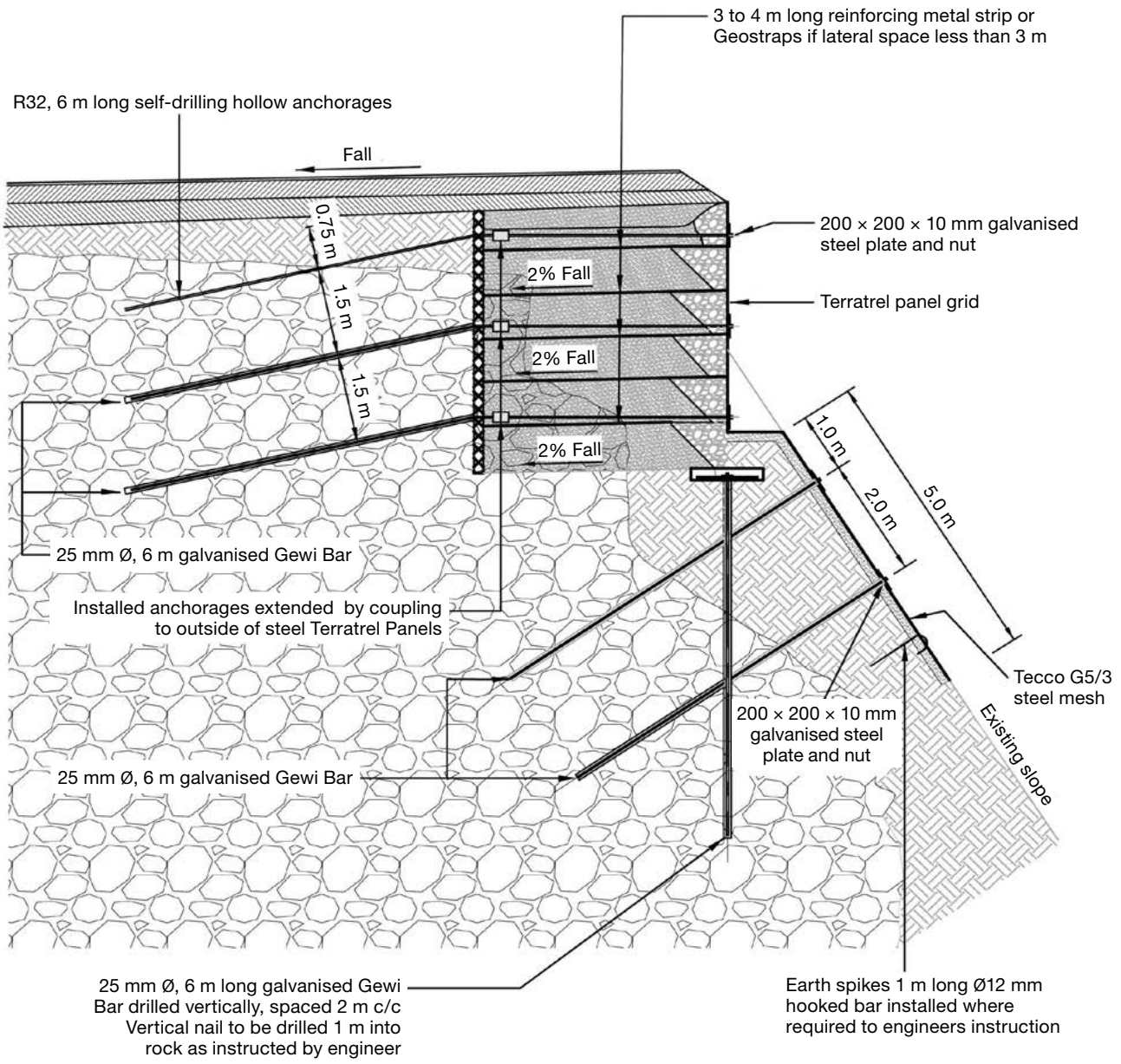

Figure 3: Typical section through the tied back reinforced embankment

Figure 4: Installation of rock bolts by Penny-Farthing

Figure 5: Road widening completed to subgrade level

used, due to the variable soil/bedrock, a TerraLink® system was decided on. In this case the Reinforced Earth structure was not used as the primary earth retaining system, but was rather used to connect the facing elements to the primary soil reinforcements to create a vertical facing and widen the road shoulder. The TerraTrel facing was connected to the back face with a Geostrap®.

CONSTRUCTION CHALLENGES

The variable and localised nature and orientation of hard rock outcrops necessitated the need to implement steel strips and Geostraps to ensure the internal stability of the Reinforced Earth system.

It was a concern during construction that the steel TerraTrel panels were prone to slight lateral deformation, which could have led to vertical settlement of the road layerworks and deformation to the surfacing. Rock bolts (25 mm and 32 mm diameter galvanised bolts), used for temporary stability during excavation, were extended through the Reinforced Earth embankment and fixed on the outside of the steel TerraTrel panels by means of a 200×200 head plate and nut. By nominally tensioning the rock bolts, the TerraTrel became an extension of the 200×200 head plate, thereby providing further lateral stabilisation and reducing the TerraTrel deformation.

Founding conditions on steep slopes will always have inherent issues to overcome and, although the bearing pressures were relatively low, the founding conditions on the slope varied from loose soils to solid rock, which would certainly have led to differential settlement along the front face (TerraTrel panels). In order to create a uniform and stable foundation for the system, vertical rock bolts were installed 1 m into the rock to exploit the enhanced end-bearing characteristics as 'mini-piles'. These 'mini-piles' were fixed to a continuous reinforced beam, so that the system could be perceived as a continuous piled beam raft.

It was evident at the start of the works at Wall 3 that erosion below the toe of the new Reinforced Earth system could potentially lead to localised slope failures below the wall, so an erosion protection and stabilising mesh from Geobrugg was installed below the new Reinforced Earth wall system. Due to the current drought conditions in the Western Cape, it was decided to delay the revegetation of the lower slope.

CONCLUSION

The importance of retaining the geotechnical engineer during construction phases is an effective method of managing the subsurface risks. This is even more evident in the 'design and construct' contract where contractual risk shedding does not indemnify the project from geotechnical issues or reduce the geotechnical uncertainty.

This project demonstrated that a design team in regular communication with one another can manage the geotechnical risk and deliver a successful project.

John Versfeld and his 16 labourers would probably agree that the latest works in this pass have contributed another chapter to its history. ◻

Figure 6: Aerial view of completed road widening embankment; note that the revegetation of the slope below Wall 3 still has to be undertaken (Photo: Paul Fairbrother)

PROJECT TEAM	
Client	Western Cape Government: Department of Transport and Public Works
Consultant	Gibb (Pty) Ltd
Sub-consultants	Kantey & Templer (Pty) Ltd
	Reinforced Earth South Africa (Pty) Ltd
Main contractor and sub-contractor	Civils2000 (Pty) Ltd
	Penny-Farthing (Pty) Ltd
Project value (Wall 3 only)	R8.4 million

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• No. 1 Beachy Head	Plettenburg Bay, WC	361m^2 of lateral support with 119 forum bored piles
• UWC Sciences Building	Bellville, WC	129 CFA piles
• Water and Sanitation Head Office	Bellville, WC	215 DCIS piles
• Soetwater & Karusa Wind Farm	Matjiesfontein, WC	1439m of geotechnical drilling

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The problem with MSE walls – a case study in support of integrated geotechnical engineering design

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INTRODUCTION

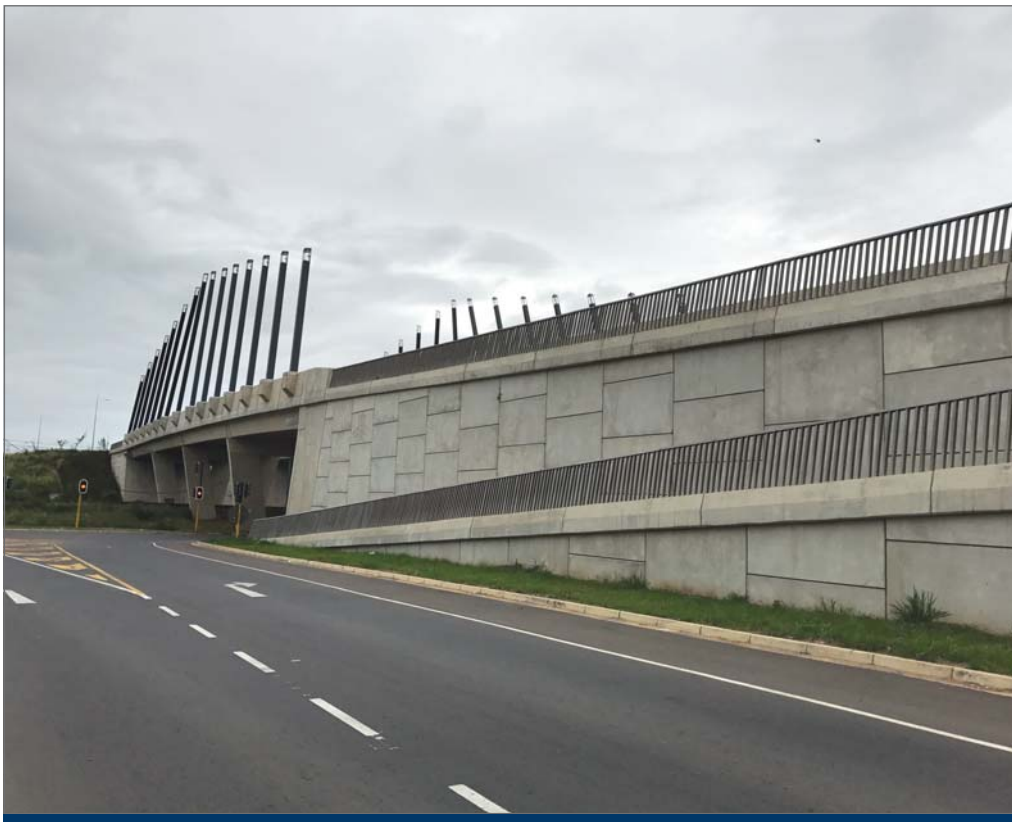
Mechanically Stabilised Earth (MSE) walls have a growing application in place of conventional retaining systems for varying reasons, most notably economy and constructability. However, there have recently been a number of failures or instances of poor performance of these systems throughout the southern African region. An evaluation of these indicates that there are two fundamental causes for poor performance. The first relates to the nature in which MSE structures are planned, designed and constructed. The second relates to the need for the geotechnical designer to develop a clear understanding of the subsurface conditions, together with a need for routine verifications of the ground conditions, design, construction and materials during the process of construction. This article presents a case study of the planning, design and construction of an MSE wall, in this case a Reinforced Earth® wall, which was successfully constructed over a poor subgrade in Durban. In the context of the preceding discussion, the case for integrated design by geotechnical engineers is made, given the uncertainties with the "design and build" procurement model which is typically used for the supply of MSE systems.

MR458 ROAD-OVER-RAIL BRIDGE

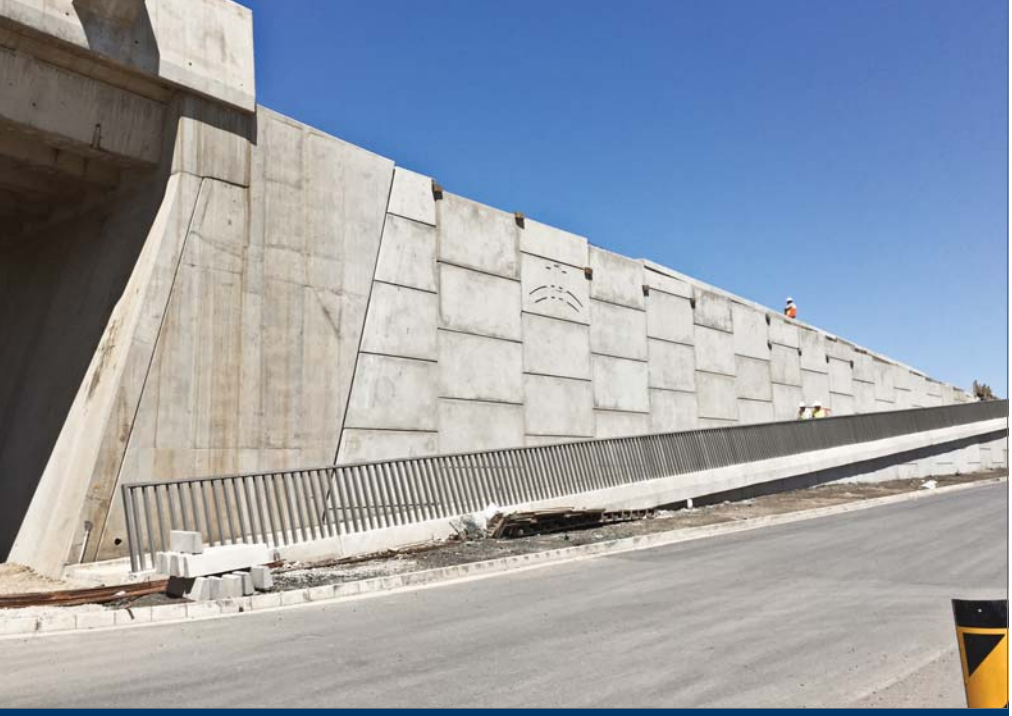
Located approximately 20 km north of Durban, connecting JG Champion Drive to the Cornubia Industrial and Business Estate (CIBE), lies the new MR458 road-over-rail bridge which leads motorists directly to the main entrance of the estate. CIBE forms part of the Cornubia development, which is a multi-billion rand integrated human settlement incorporating industrial, commercial, residential and open space use. It is being developed by Tongaat Hulett Developments and the eThekwini Metro Municipality, and has been adopted by the Cabinet as a national priority project.

The dual bridges, each 88.5 m in length, comprise four spans with slanted piers and an integral deck consisting of pre-tensioned beams. The abutments to the bridges are also slanted and include some 200 m of MSE wall (MSEW) approach fills. The site is situated in an alluvial plain, with the western portion of the site previously cultivated as a watercress farm, while the eastern portion was used as a dump site for sludge from a nearby wastewater works.

The site offered poor founding conditions for the MSEW and bridge structures, with over 120 mm of settlement



Completed Cornubia Bridge and MSE walls – MSEWs are the ultimate geotechnical structures, as they have an incontrovertible link with the soils within, around and below



Cornubia Bridge west abutment – the abutments were slanted, which would have visually accentuated the effect of any downward settlement behind the abutment

Ground improvement comprising dynamic replacement craters

Load transfer platform designed in accordance to EBGEO

predicted for the 9.5 m high abutments. The bridge abutments are buttressed walls, leaning back at a 1 in 4 slope. The fact that the abutments were slanted implied that any settlement between the bridge (which is founded on rock-socketed piles) and the MSEW would be accentuated and immediately visible in the architectural feature of the bridges. The development of negative skin frictions on the piles for the abutments was also a concern, due to the poor subgrade. Dynamic replacement stone columns were thus required to improve the founding conditions below the MSEW.

OVERVIEW OF MSE WALLS

MSEWs in the broadest sense comprise concrete block or panel façades connected with multiple layers of inclusions acting as reinforcement in the soils placed as fill. The complexity begins with the wide array and ever-developing soil reinforcement technologies, suppliers, materials and even connection and construction methodologies, which implies that the performance of any MSE system hinges on the application for which the system is selected.

Added to this, is the multifarious interaction that occurs with the founding soils *on* which it is built and the soil materials *with* which it is built. MSEWs are the ultimate geotechnical structures; the structure has an incontrovertible link with the soils within, around and below. The stresses developed and strain encountered in the soils and the structural system itself, influence each other and cannot be designed independently of each other.

The case for the use of the MSE systems can be strongly motivated when the cost and time taken to construct such systems are considered. Cost benefit comparisons undertaken on recent bridge projects have shown that only once walls exceed 20 m in height should consideration be given to the replacement of sections of MSEW with additional bridge spans if there are no other considerations affecting the bridge length.

An inherent problem with MSE systems is the way in which they are procured, planned, designed and constructed. In trying to account for the varying patented systems, the most common approach is to allow the contractor to supply the design based on usually very limited information by the designer. For example,

drawings could very simply state "wall design by others" with limited or inappropriate specification, and no guidance on design, design specification or parameters. This option is selected to encourage competition between suppliers, and is generally welcomed by suppliers, despite the self-inflicting problems caused.

Consequently, this procurement model has led to the following unintended consequences which undermine the effectiveness and credibility of the MSEW system:

- The elimination of proper geotechnical investigation and almost no involvement by geotechnical engineers as part of the principal design or owner's consulting team. This is supposedly under the assumption that the contractor (or supplier) assumes the risk, which is not the case.
- Without specifications provided by the owners, or if such specifications are too broad, it is not possible to compare MSEW on any basis other than on price. Hence, the least robust design will produce the lowest cost. Lack of a design basis memorandum may result in liberal soil strengths, optimistic loading conditions, and favourable groundwater conditions. Additionally, some proprietary design approaches eliminate or alter minimum standards of practice (e.g. facing connection, bearing capacity, corrosion protection, internal failure surface orientation, and global stability) (Simac *et al* 2007). This is particularly true in South Africa, where even unreinforced block retaining walls are confusingly marketed as technically equivalent to MSEWs.

Returning to the issue of risk, and with reference to the GCC and COLTO specifications, the design consultant retains responsibility for overall stability and design criteria. The system thus actually involves a shared design responsibility between the owner, design consultant, supplier and/or contractor.

It is not a design-and-build method, which is the method that the owner and designer usually believe they are getting, even though none of the procedural, contractual and legal criteria necessary to invoke a design-and-build scenario are put into place. This leads to confusion before, during and after construction regarding exactly which party assumes engineering responsibility (and liability) for important design decisions and quality assurance.

The above highlights that there is significant responsibility assumed by the owner and his designer when specifying MSEW systems, and in the case of the new Cornubia Bridge, the poor subgrade conditions and a bridge design which accentuates any settlement of the MSE walls behind the abutment, necessitated that an integrated geotechnical design approach was required for the success of the project. This integrated geotechnical design entailed the following:

- **Geotechnical investigation** in accordance with the SAICE Geotechnical Division Code of Practice for Site Investigations, and led by a registered geotechnical engineer;
- An **initial detailed design** to establish technical and performance criteria, and select suitable MSEW systems;

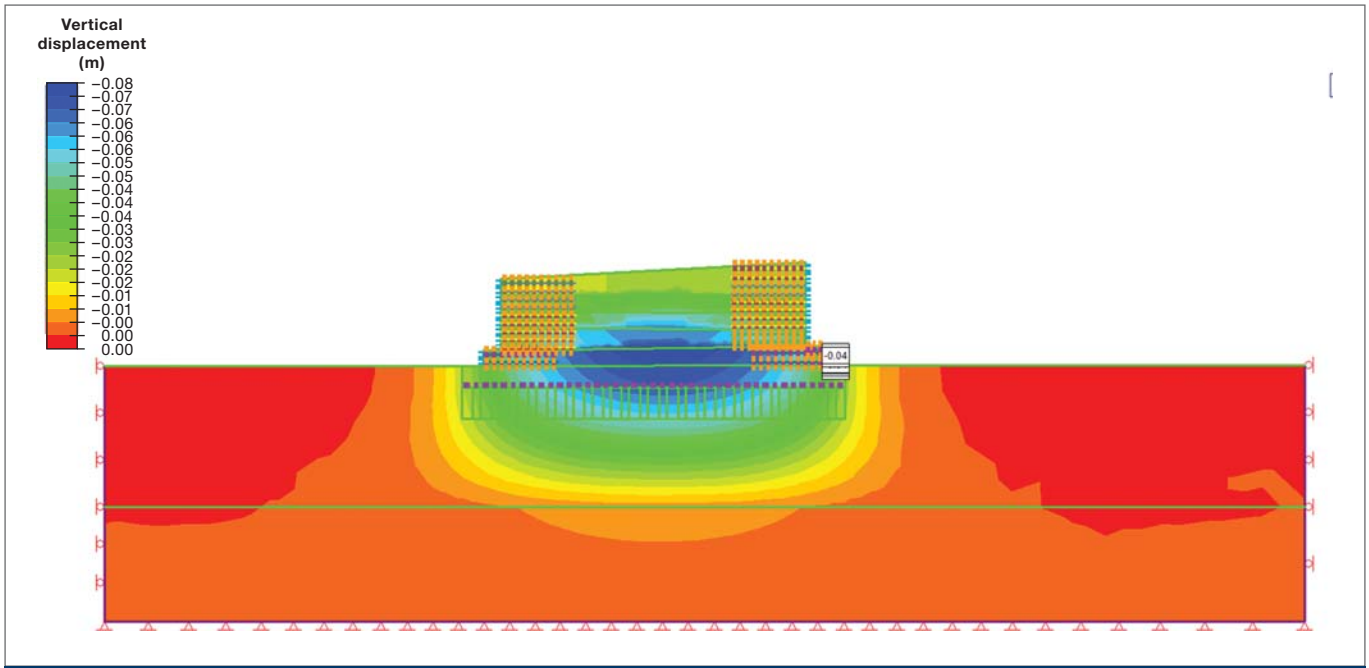


Figure 1: Staged finite element analysis – the analyses corresponded well to the surveyed settlements measured on targets placed on the MSEW panels

- Development of **tender and design specifications**, appropriately considering the various design constraints;
- **Finalisation of design** and design interfaces, with consideration of the final selected MSE system (by winning tenderer, Reinforced Earth), including peer review of respective designs; and
- Implementation of an appropriate **quality assurance**, testing, construction and performance monitoring regime. This included an appropriate level of construction supervision by the geotechnical designer and the supplier.

The above steps are expanded upon further below in the context of the project.

Geotechnical investigation

A comprehensive investigation was conducted, appropriate to the geotechnical conditions and structure proposed, and adhering to the SAICE Code. Investigations were undertaken by ARQ Consulting and comprised several rotary core boreholes to depths of up to 17 m, in-situ testing (SPT and DPSH) and laboratory testing on soils and rock.

The DPSH tests showed very little resistance through the alluvial materials, with the probe progressing some 200 mm per blow in some instances. The SPT results, using cautious estimate SPT-N blow count, was in the order of 8 over the top 5 m, and SPT-N blow count of 13 from 5 to 10 m. Mudstone and dolerite were encountered at depths exceeding 10 m.

Initial detailed design

An initial detailed design was undertaken using typical and varying parameters for the MSE structure to account for different systems which could potentially be used. This required the selection of a number of performance criteria against which various MSE technologies could be evaluated, and a cost benefit and optimisation exercise. This optimisation duly considered various influencing elements – such as ground improvement on overall stability and settlement, and the availability and selection of fill material on internal stability.

Due to the poor founding soil stiffness values, associated low bearing capacity and expected high settlements, ground improvement measures were implemented for the foundations of the MSEW. This ground improvement was achieved by means of dynamic replacement with the rapid impact compaction (RIC) method. The RIC specially adapted machine uses a 9 ton weight from a drop height of 1.5 m.

The stone column raft was capped with a load transfer platform, consisting of a high-strength bi-directional geotextile, and a granular raft consisting a G6 material. The platform was reviewed in accordance to methods described in SANS 207:2006 and the Recommendations for Design and Analysis of Earth Structures using Geosynthethics Reinforcements (EBGEO). The two methods differ in the way the soils between stone columns are analysed, where the one ignores the subgrade provided between stone columns and the other incorporates the subgrade.

The MSEW consisted of a tiered walkway system with specially made panels, 3 m wide and 1.5 m high, according to the architectural requirements. The settlement performance was modelled in finite element software to establish the improvement that could be expected in the overall behaviour of the system whilst using a dynamic replacement stone column foundation with a granular platform. The offset between the two tiers implied that the top and lower tiers would influence each other. The maximum tension line would, however, be at a flatter angle when compared to a non-tiered system, and therefore strip lengths need to be reviewed for pull-out, differently to methods used for a non-tiered system.

Without any soil improvement, some 120 mm of settlement was expected due to the fill placement. The dynamic replacement stone columns and G6 platform were shown to improve the settlement behaviour in a staged finite element analysis to some 50 mm, while the estimated self-weight settlement was estimated to be negligible at some 12 mm.

Tender and design specifications

Considering the design limitations, development of a specification for the supply and internal design of the system, with due consideration of the SANS 10160 and SANS 207 requirements, was

undertaken. Specific attention was given to design safety factors, design loads, design responsibility, supplier involvement and responsibilities during construction, including verification testing.

Technical specifications thus set out minimum technical and performance criteria required by the supplier, allowing for the assessment of the tenders on adherence to criteria first, followed by price.

Finalisation of design

Although performance criteria were set at tender stage, these governed the range of several design parameters important to the design. The finalisation of the design was dependent on having discrete design values, which were only known once the final MSE system was selected (i.e. successful tender is known).

This stage also included peer review and collaboration on the design with the suppliers on their internal stability designs and compliance with specification. Similarly the supplier was able to review the owner's external stability designs. This ensured that design assumptions

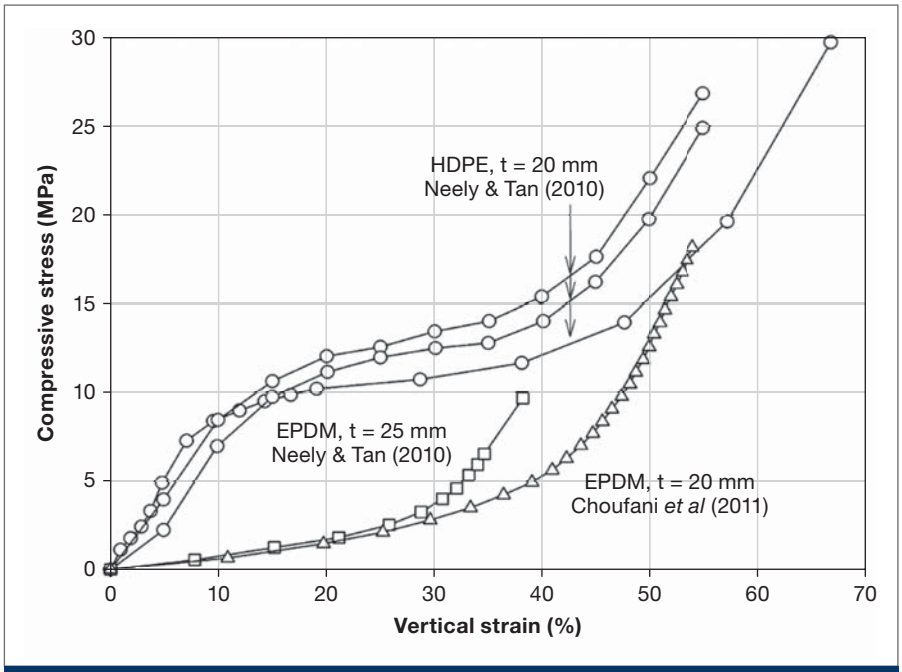


Figure 2: Compression behaviour of HDPE and EPDM bearing pad materials (Damiens *et al* 2013)

Although performance criteria were set at tender stage, these governed the range of several design parameters important to the design.

Bearing pads placed on vertical joints between panels

and interfaces were understood. One such example was the internal or inter-panel settlement:

As settlements were very much driving external design and performance criteria, it was important that this was communicated and considered in the internal design, and in particular the concrete panel façade and the larger-than-normal panels used. Bearing pads are utilised to absorb internal (as a result of settlement of the fill) and some external settlement (as a result of foundation settlement). These are placed in horizontal joints of discrete precast concrete panels in order to allow the panel and the reinforcement to move down with the reinforced fill as it is placed, and settles. This mitigates downdrag stress and provides flexibility to the façade to account for differential foundation settlements. SANS 207:2006 states that for discrete panels the vertical movement capacity of the system should be a minimum of 1 in 150 relative to the panel height.

Ethylene propylene diene monomer (EPDM) bearing pads of 25 mm were placed in the joints between the discrete panels at a spacing of 0.75 m. The expected stress-strain relationship is provided in Figure 2 for three different types of bearing pads.

The foundation and internal settlements, panel rotation and weight of the panels were all modelled in geotechnical finite element software to analyse the number of bearing pads required on the vertical joints between panels. Additionally, the loads in the steel strips were reviewed to compare them to the capacities they have been designed for and to establish if additional or higher loads are attracted to the strip at the facing-strip interface (T_{conn}).

Quality assurance

The final and most important stage of a fully integrated geotechnical design is the implementation of an appropriate quality assurance, testing, construction and performance monitoring regime. This must include an appropriate level of construction supervision and oversight by the geotechnical designer.

For the ground improvement, quality assurance was undertaken by reviewing the continuous number of blows versus penetration plots, plate load tests, DPSH and continuous surface wave (CSW) testing, which all provided verification of the subsoil conditions and the performance of the ground improvement. This was in addition to standard quality assurance and testing of the concrete, backfill, layerworks, materials supplied, survey, line and levels.

Settlement was monitored on panels and in the roadway, and limits set which governed the timing for placement of final road layer works and ancillary features.

CONCLUSION

It is essential that professionals charged with the responsibility of planning, designing and implementing MSE retaining systems understand the application, limitations and costs associated with such technologies, which are ever developing and advancing. This responsibility is often exacerbated by difficult subsurface conditions, restricted right-of-way and marginal sites with challenging topography, variable climatic conditions and other environmental constraints.

The notion that in projects where public money is involved these systems are procured on a design-and-build basis, thus absolving the owner and/or consultant of any responsibility, is not correct. Notwithstanding the fact that the contractual mechanisms enabling a design-and-build approach are seldom put into place, there are highly complex interactions between internal and external design factors, and between the soils and structural members making up the MSE system. If failure occurs, the responsibility will invariably need to be shared, as it will always be difficult to identify a single causational factor which led to the failure.

Furthermore, the recent trend by owners and consultants of providing contractors with limited or inappropriate investigation, design specifications and parameters, and performance criteria, unfairly jeopardises the entire industry. It is a clear dereliction of design responsibility.

Whilst it is understood that both COLTO and SANS 207 are currently under review, limitations to these will remain. The requirement for adequate ground information and an integrated approach to the geotechnical engineering design of MSE represents best practice and reduces the risk for all project participants. Likewise, the introduction of new and conflicting technologies implies *more* involvement of geotechnical design engineers in defining the problem and levelling the playing field, not less.

ACKNOWLEDGEMENTS

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REFERENCES

A full list of references can be provided by the authors on request. ◻

PROJECT DATA	
Client	Co-funded by eThekwini Municipality (60%) and Tongaat Hulett Developments (40%)
Consultant	SMEC South Africa
Contractor	Fountain Civil Engineering (FCE), with Reinforced Earth as MSE supplier
Project value	R145 million

Ground improvement by compaction grouting in IHC5 and IHC7 dolomitic conditions

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INTRODUCTION

AMKA Products is a black-owned and managed South African enterprise in the health and beauty industry, led by several generations of the Kalla family since its inception in the 1950s. It has grown from a humble trading operation struggling to break into the mainstream retail sector to now having a strong presence in South Africa, and showing increasing growth into Africa. This Top 500 South African company is consistently rated as one of the top ten empowerment companies, and was the winner of the Shoprite Supplier of the Year award for 2016, and the recipient of a PMI Diamond Arrow Top Manufacturer award in 2013.

Today the company employs more than 1 000 staff members and produces over 800 fast-moving consumer goods products which are marketed in the hair care, skin care, fragrance and home care markets, with over 30 leading brand names sold in 35 African countries. The company operates, perhaps a little disjointedly, from six manufacturing plants in Sunderland Ridge, Pretoria, some of which it has outgrown, necessitating a sizeable expansion in its operation – currently under construction – to cope with present and future demand.

This expansion project has provided AMKA with an ideal opportunity to both consolidate and streamline the future operation into an integrated, state-of-the-art manufacturing and warehousing enterprise, following global best practice procedures in the industry.

EXPANSION PROJECT

For the purposes of this expansion, AMKA procured a prime site on the corner of the M10 and R55 arterials in Sunderland Ridge (see Figure 1), and proceeded with the construction of the first of the warehouses – denoted as CDC1 in Figure 1 – roughly 20 years ago. This warehouse is presently fully functional and is operated under contract by Imperial Logistics.

The second and significantly larger phase of the expansion project commenced some two years back, with a view to construct the first of two new factories on the same premises, which will supply both the existing (CDC1) and future (CDC2) warehouses, supported by a formal multi-storey office block with upgraded access control.

The new CDC2 warehouse has been designed to comprise racks standing approximately 50% higher than the present 9 m high racks of CDC1, and will be used for longer-term storage than CDC1, which has a very high stock turnover.

Figure 1: The AMKA factory and warehouse site on the corner of the M10 and R55 arterials in Sunderland Ridge

SITE CHALLENGES

With the site dipping from northeast to southwest at roughly 1:18, the client opted to terrace the site in preparation for the expansion, with CDC2 and the office structures located on the upper terrace, adjoining CDC1, and two phases of the factory component of the operation located on a lower terrace – some 8 m lower than CDC2 – to optimise materials production, handling and distribution.

These terracing operations necessitated lateral support being applied to the 8 m high cut slopes, which took the form of a mesh-reinforced and shotcreted soil-nailed wall, half of which was treated as a permanent structure to support CDC1,

with the remainder a temporary measure to facilitate the safe construction of a new reinforced concrete retaining wall, which is integrated into the factory warehouse interface structure.

In addition, the entire fire protection system for the expanded facility is being significantly upgraded and, because the terracing operations have dropped ground levels below the present outfall sewer, a new pump station is being constructed to pump the effluent up to the nearby wastewater treatment plant.

GEOLOGICAL SETTING OF THE PROJECT SITE

Sunderland Ridge, as with large tracts in the southern half of the greater Tshwane metro, is underlain by a dolomitic soil/rock profile, with all of the inherent flaws and challenges associated with karstic dolomite.

Early development on karstic dolomite generally became characterised by significant sinkhole formations, which led to extensive geological and geotechnical engineering appraisal to determine the triggering mechanisms and influencing factors, and the application of severe restrictions on development. As a result, large tracts of land in the greater Centurion area have been left undeveloped until the present generation, when pressure on the city expansion simply necessitated a reassessment of the blanket ban on these previously off-limit areas, and the development of appropriate risk mitigation strategies.

In the interim, too, much has been learned and documented about dolomite – particularly within the Council for Geoscience and the Department of Public Works (DPW). The DPW, who administers extensive property on dolomitic land and had to contend with frequent sinkhole/subsidence damage, has taken a notable role in developing guidelines and methodologies to regulate and mitigate the risk of construction in dolomitic areas.

INTRODUCTION OF SANS 1936:2012 (DEVELOPMENT OF DOLOMITE LAND)

These guidelines, which previously took the form of various documents produced between 2005 (National Home Builders Registration Council), 2007 (Council for Geoscience) and 2010 (Department of Public Works), and which were based on more than five decades of geological and geotechnical engineering experience, were recently codified into a national Code of Practice – SANS 1936 of 2012 – by the various stakeholders in the broader geotechnical profession.

Under this new code all developments on dolomite are now obliged to comply with the stipulated requirements. Distilling the code, for the more extreme (D3 and D4) karstic dolomite sites, the following appears to be the overarching structural design requirement:

"In proposing suitable foundation types in D3 and D4 areas, consideration shall be given to the potential loss of support which could be anticipated for the designated inherent hazard class based on expected initial sinkhole size. The philosophy to be applied to the design of the foundations is that there shall be sufficient structural integrity and stability to allow occupants to safely escape in the event of sudden loss of support below the foundations of a structure."

AMKA FOOTPRINT INVESTIGATION

In the interlude between the construction of CDC1 and the present phase of the expansion project, the rules governing developments on dolomite were formally codified, whereby significantly more responsibility was transferred onto the developer to mitigate the effects of dolomite instability.

Whereas in the past – even for CDC1 – the investigation and design requirements were less stringent, now, under the new code, compliance has become mandatory to obtain the necessary approvals for appropriate development from the City of Tshwane (Ekurhuleni on the East Rand is similarly affected), under the watchful eye of the Council for Geoscience, for all development in dolomitic areas.

In compliance with the code, Crossman Pape & Associates (CPA) were appointed to provide formal preliminary and footprint geotechnical investigations and dolomite stability assessments for the remaining portions of the AMKA site earmarked for development. Based on their findings, the project site was predominantly classified as IHC5 (high risk of small sinkholes, typically < 2 m in diameter), with a localised IHC7 sector (high risk of large sinkholes, up to 15 m in diameter) encompassing a portion of the northern third of the site – rather fortuitously earmarked for the main office and gatehouse structures.

Prior to 2012 it was unlikely that permission would have been granted to develop any structures in the IHC7 portions of the site. Under the new code provision is, however, made for development of even the harshest dolomitic conditions (up to IHC8) – designated D4 developments – subject to compliance with the code requirements, most pertinently that the structural foundation and ground

Figure 2: The heavy surface bed punctuated by free-standing concrete columns supporting the entire warehouse structure

improvement designs are undertaken and monitored for compliance by a so-called *Competence Level 4 geo-professional*.[1]

In the present instance the engineering design and site supervision aspects of this project were referred to Alastair Morgan Pr Eng (Technical Director at Geoid Geotechnical Engineers) who, amongst a small fraternity of experienced geotechnical engineers of a similar generation, has the requisite Level 4 accreditation to undertake D4 design and review work.

STRUCTURAL DESIGN

From a structural design perspective, the requisite *loss of support* criterion may be considered as:

- provided for by appropriate structural (foundation) members, and
- mitigated by making use of appropriate ground improvement techniques, in combination.

As such, there may be a cost benefit trade-off between *ground improvement* and *structural rigidity of the foundation*, depending on the particular circumstances.

In the present instance, the structural members necessary to span a 15 m *loss of support* in the IHC7 zone were assessed by structural engineering company EDS to be prohibitively large for the affected portions of the office block structure. A 5 m *loss of support* was selected as the maximum void which could reasonably be accommodated in the structure, and the sector identified for major ground improvement as described below, to treat the perceived highly voided founding environment.

The adjacent sectors of the CDC2 warehouse on IHC5 land comprise the heavy surface bed punctuated by nine free-standing concrete columns supporting the entire warehouse structure (see Figure 2 looking across the warehouse footprint towards the office structure).

A key issue raised in the design review was the critical potential *loss of support* beneath the rows of warehouse racks in CDC2 which exert very high point loads on the surface bed. It was reasoned that even a small loss of support beneath the surface bed could translate to significant tilting of the proposed 15 m high racks, with collateral damage occasioned by knocking over adjacent racking in a domino effect.

As such, the scope of the ground improvements was extended from simply the structural foundations alone, to include the footprint of the entire warehouse floor, with particular attention paid to the major warehouse columns (illustrated in Figure 2).

Despite the fact that most of the warehouse is interpreted to fall within an IHC5 zone, the stability of the surface bed necessitated a conservative view being taken of this comparatively more favourable zone, in the knowledge that the hazard classification is, in reality, based on limited advance information.

GROUND IMPROVEMENT

Several ground improvement techniques were considered for this site, and weighed up against the environmental considerations and the impact on the existing operation. Given the sensitivity of the existing CDC1 warehouse – which does not evidently have any ground improvement applied, due to the less onerous regulations of the past – and the vulnerability of the racking to loss of support or movement induced by heavy vibrations, as well as the risk to the soil-nailed lateral support for CDC1, all forms of dynamic compaction in close proximity to the warehouse were effectively eliminated as being untenable.

Under the imposed constraints and risk of potentially metastable karstic conditions beneath CDC1, the professional team were of the view that compaction grouting presented the only truly viable solution – particularly so since the client's brief was that nothing should hinder or place at risk the CDC1 operations, which needed to remain fully functional and essentially unaffected for the full duration of the adjacent construction.

The compaction (or low mobility) grouting process is designed to intersect and fill the disseminated voids and cavities which occur in the dolomite residuum, i.e. essentially above bedrock level, with a view to interrupting or inhibiting the sinkhole-forming mechanism – but not intended to fill the massive caverns assumed to occur within the dolomite rock mass at great depth. It essentially involves drilling a pilot hole, using a rotary percussion drilling rig through the dolomite profile, nominally 5 m into the proven bedrock, to prove the competent rock horizon. A low-mobility grout – nominally 10 MPa cement:sand grout mix with a slump of around 100–130 mm and a consistency of toothpaste – is injected in an upstage sequence from the bottom of the hole, under pressure of between 2–3.5 MPa. The operation is undertaken across the site in a grid pattern, utilising primary, secondary and, where necessary, tertiary (and subsequent) stages to progressively 'seal' the voids to the designer's requirements.

RODIO Geotechnics (Pty) Ltd were appointed to undertake the grouting operations, which commenced in June 2016, under the direction of the geotechnical specialist.

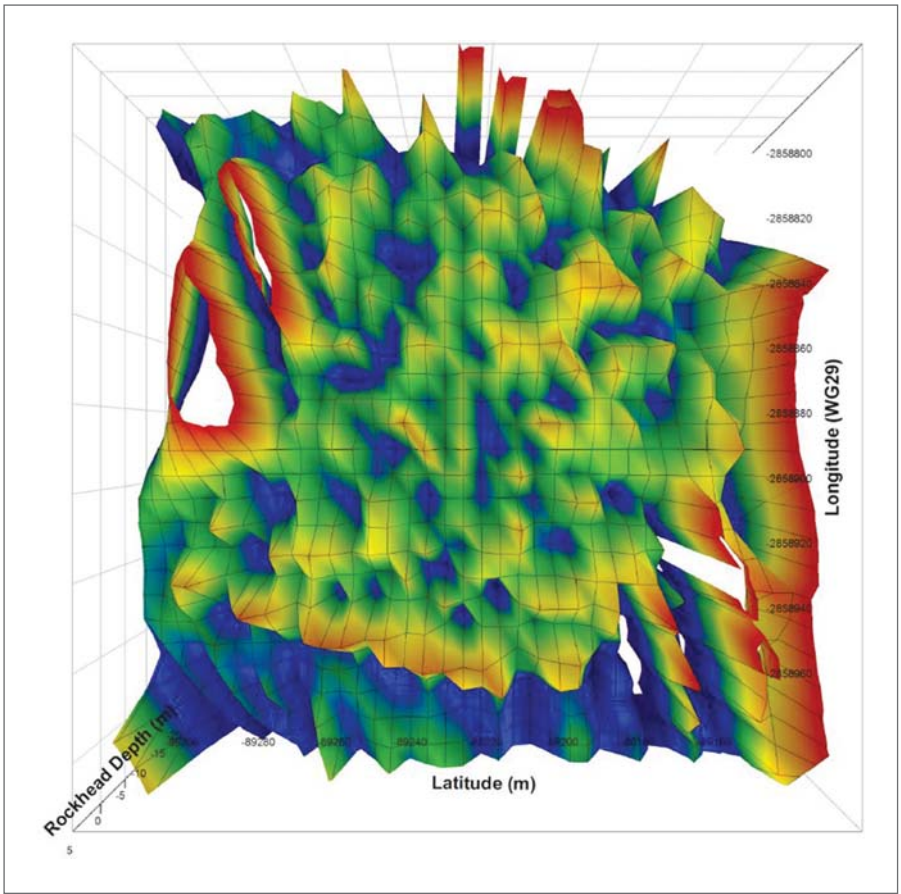

Figure 3: A normalised 3D surface of the dolomite rockhead

GROUND PENETRATING RADAR

In view of the anticipated cost implications of the grouting, experimental use was made of ground penetrating radar (GPR) to scan the site from ground level, from which a composite, georeferenced 3D image was generated of the relative density of the ground, in an attempt to expose horizons exhibiting possible cavities and/or significant solution channels, which could then be specifically targeted for treatment and rehabilitation.

Although the initial results appeared to hold much promise for predicting problematic areas and targeting the compaction grouting operations, very little correlation between grout volumes and inferred voided and disseminated cavity areas was proved in practice, and in the end little confidence could be placed on either the reliability or the value of this investigation technique, which is rather unfortunate.

Perhaps, as the technology improves with time, these limitations may be overcome, but at present we consider the techniques to be unreliable and of limited value.

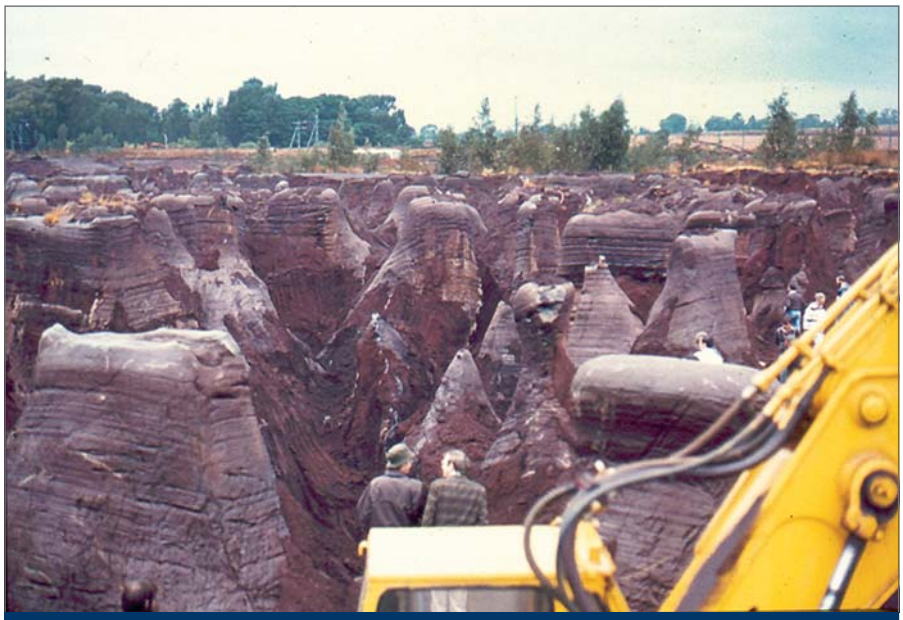

Figure 4: The local dolomite bedrock morphology is inferred to follow a similar mosaic to that exposed in the Lyttleton dolomite quarry (pinnacles) (Photo: Dr Peter Day, Jones & Wagener)

DOLOMITE OBSERVATIONS

On the basis of a rudimentary assessment of the incomplete, but comprehensive, dataset of approximately 1 200 compaction grouting boreholes drilled to date, a normalised 3D surface of the dolomite rockhead is presented in Figure 3.

Our preliminary observations are as follows:

- The local Sunderland Ridge dolomite bedrock morphology is inferred to follow a very similar mosaic to that exposed in the Lyttleton dolomite quarry, illustrated in Figure 4.
- The mosaic comprises numerous relatively steeply-sided pinnacles – many of which protrude above the reduced platform ground level – interspersed by deep troughs which, on this site, are typically no more than 20 m deep.
- Infill material comprises typical chert rubble with relatively limited traces of the low-density, porous and problematic WAD (weathered altered dolomite).
- As with the Lyttleton quarry, the pinnacle formation is relatively random, with no appreciable pattern, other than perhaps several sets of preferential troughs on an ill-defined, but nominally northwest-southeast axis.
- The projection of these pinnacles, interspersed with troughs of thick chert rubble infill, was well exposed during the soil-nailing lateral support installation in the cutting immediately adjacent to CDC1, shown in Figure 5.
- These pinnacle protrusions were typically measured to occur at between 2–6 m centres, in keeping with the dolomite hazard class previously interpreted for this portion of the site.

COMPACTION GROUTING DESIGN

In view of the high costs of ground improvement by grouting, an iterative procedure of progressively refining the

grouting resolution was adopted to optimise the grouting, rather than simply working systematically from one end of the site to the other and grouting each and every grid node. This process involved drilling and grouting every alternative node on an 8 m primary grid – which corresponded with the column/ground beam positions of the structure – to provide an overview and coarse model of the prevailing conditions, and a reassessment of the inherent hazard class to justify subsequent work.

In all instances, the alternate primary positions, previously skipped, were drilled and grouted on the second pass, refining the model, and in so doing providing support nodes on a regular 8 m grid. Although this roughly halved the potential void, it did not yet meet the requisite 5 m *loss of support* criterion.

A more selective secondary grid – drilled on the diagonal midpoints between the primary holes – was undertaken where the grout take in the neighbouring cluster of four primary holes exceeded a selected threshold of 10 x the volume of the drilled boreholes, i.e. $10\Sigma\pi r^2 h$.

The grouting operation was modelled on a daily basis using ArcGIS spatial-database software, which then provided the rational basis for subsequent grouting and/or grout node elimination.

On completion of the secondary nodes, a further tertiary iteration was executed on its diagonals using the same $10\Sigma\pi r^2 h$ criterion for neighbouring nodes.

In view of the significant structural importance of the main warehouse bases, sited in the IHC5 sector of the site – for which absolute stability was essential – all of these foundation bases were subsequently audited using two additional grouted boreholes on the primary axis exhibiting karst formation. In two of the nine bases the audit provided clear evidence of large inter-connected voids, despite the ground improvement already applied, which were then spot-treated with very-high-resolution perimeter grouting to ensure adequate support of these critical foundations.

GROUTING OBSERVATIONS

Taking an orthogonal view of the same idealised bedrock surface as an underlay for the compaction grouting dataset, the composite image shown in Figure 6 is generated, from which the following preliminary observations are made:

- At an elementary level, there appears to be no appreciable difference in the dolomite morphology between the IHC7 (office) and IHC5 (warehouse) portions of the site.
- Notwithstanding this, the grout takes (i.e. grout volumes consumed during injection) in the IHC7 zone are appreciably higher than those in the IHC5, which supports the original assessment of the two hazard class zones on this site.
- As would be expected, higher grout takes typically correspond with a deeper rockhead – blue zones versus the shallow red zones.
- Notwithstanding this, significant grout takes also occur in the transition zones between the shallow and deep dolomite, which are interpreted to be the steeply-sided perimeter of the pinnacle formation.
- Portions of the site underlain by shallow bedrock are not, however, a guarantee of problem-free bedrock, a case in point being where the two most severely impacted support columns for the warehouse (light blue base outline in Figure 6) are underlain by shallow dolomite rock.

Figure 5: The projection of Lyttleton-type pinnacles, interspersed with troughs of thick chert rubble infill, was well exposed during the soil-nailing lateral support installation in the cutting immediately adjacent to CDC1

- This seeming anomaly may possibly be attributed to the prominent steeply sloped sides of the pinnacles which, we interpret, render it highly susceptible to WAD and cavity formation, or alternatively the presence of a shallow 'throat' feature in shallow rock, either of which may be responsible for the high grout takes.

CONCLUSIONS

Despite the greatly expanded scope of work, largely brought about by the need to support the warehouse surface bed in addition to the key foundations – an item which was essentially not fully budgeted for – the ground improvement designers were constrained to do whatever possible to protect the budget against significant cost overruns.

As the geotechnical specialist was appointed on a design-as-you-construct basis, an iterative grouting methodology was adopted, comprising initially broad concentric circles of progressively concentrated grouting, rather than a simpler sequential operation from one side of the site to the other, drilling all possible nodes in the process.

This procedure did, however, require that the ground improvement works had to be scheduled with a reasonable head-start preceding and accommodated by the main contractor, GD Irons Construction, with provision for progressive release of sectors of the site as the treatment was completed.

The methodology adopted in the above manner provided an excellent opportunity to model the site conditions holistically on a daily basis, enabling the designer to critically evaluate the need, or otherwise, for subsequent work within the broader context of the site and the local foundation support requirement.

Based on this progressively refined model, the designers were able to eliminate a vast number of unnecessary secondary/tertiary remedial work in areas exhibiting comparatively favourable conditions, which furthermore provided the rational basis for localised detailed perimeter grouting of the foundation bases where required.

Using the modelling techniques as discussed, the design is interpreted to have satisfied not only the requisite 5 m *loss of support* for the structural foundations, but also the greatly increased support of the warehouse surface beds – within the original budget and without compromising the original scope of work.

ACKNOWLEDGEMENTS

The authors of Geoid Geotechnical Engineers would like to thank AMKA Products for both the professional appointment to undertake this project on their behalf, and the permission to publish these preliminary findings within this article. ◻

NOTE

1. Geo-professionals in Competence Level 4 shall, in addition to the minimum of five years practice as experienced geo-professionals, enjoy recognition by the profession as specialist geo-professionals, possessing a level of specialist knowledge and experience above that expected of the profession. They should be making a contribution to the state of practice of the development of dolomite land by the application of advanced techniques or by means of research, publications or involvement in engineering education.

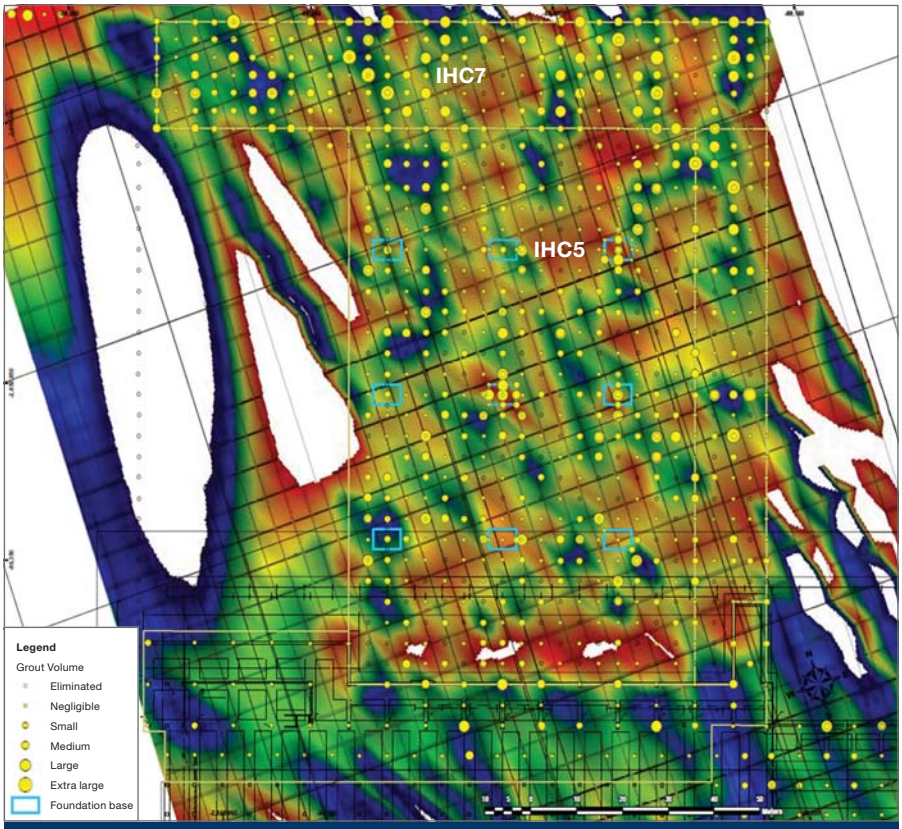

Figure 6: Portions of the site underlain by shallow bedrock are not a guarantee of problem-free bedrock, a case in point being where the two most severely impacted support columns for the warehouse (light blue base outlines) are underlain by shallow dolomite rock

A case study illustrating the advantages of detailed gravity surveys in dolomitic terrain

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INTRODUCTION

Areas underlain by dolomite, a soluble rock, are subject to the development of karst features, such as sinkholes. These pose a significant hazard to property and may even be life threatening. Currently the method used to investigate dolomitic land follows standards detailed in SANS 1936 (2012), Parts 1 to 4 (References 1–4). The following factors are used to evaluate the degree of hazard associated with sinkhole and subsidence development:

- Mobilising agencies, most importantly ingress water from leaking services or ponding of water on surface
- Bedrock morphology, significantly the bedrock pattern, involving the wavelength and amplitude of pinnacle and gryke development
- Presence of cavities and fissures, and their depth
- Nature of the blanketing layer, including its potential to erode into underlying cavities and its potential to absorb or reduce the velocity of water flowing vertically through it
- Depth of the present groundwater level and its position relative to bedrock and overburden.

Gravity surveys are the most frequently applied geophysical method for investigating dolomitic terrain in South Africa. It has become the norm to apply a station spacing of 30 m when carrying out gravity surveys, even though it is well understood, and even recommended, that the station spacing be related to the depth to bedrock. The avoidance of doing this is largely due to the competitive environment in which these surveys are conducted, as well as a lack of prior knowledge of site conditions. Previous work by the authors (References 5 and 6) has shown the value of a closer spacing when the rock head is shallow. This more detailed survey may have to be executed as a second phase of work when the extent of shallow rock head has been mapped, or after a decision on the footprint of a specific structure has been made. As this case study shows, the results of a more detailed study can bring meaning to a diverse set of drilling results, and perhaps a more appropriate classification or utilisation of the site.

RELATIONSHIP BETWEEN GRAVITY AND BEDROCK MORPHOLOGY

Gravity surveys require the collection of gravity readings (observed gravity) along with determinations of differences between stations in elevation and latitude. The calculated relative Bouguer values are then separated into residual and regional components, where regional is defined as longer wavelength changes in gravity that are of little interest to the study being undertaken. For a first approximation, the changes in residual gravity are attributed to variations in overburden thickness. This assumption is often sufficient, because of a generally large density contrast between the dolomite bedrock and overburden, which is commonly either a mix of, or single, dolomite residuum, weathered Karoo Supergroup sediments and residual intrusive.

At some stage in the survey there is a reconciliation between residual gravity values and point samples of bedrock depths derived from drilling. This may result in the derivation of a new regional-residual separation of the Bouguer field, and usually the acceptance that the particular gravity data set does not resolve all features of an assumed karstic bedrock topography. The overall bedrock depth usually influences the categorisation of the site stability.

Gravity surveys require the collection of gravity readings (observed gravity) along with determinations of differences between stations in elevation and latitude.

There is a proportional relationship between the detail, or frequency, of bedrock variations recorded in a gravity map and the station spacing employed for the gravity survey – the closer the station spacing, the more the detail that can be mapped. (This relationship is also dependent on the depth to bedrock compared to the magnitude of the changes in the bedrock head, but here we are considering areas where variations in bedrock head are sufficiently large to noticeably affect the gravity field.) There will always be

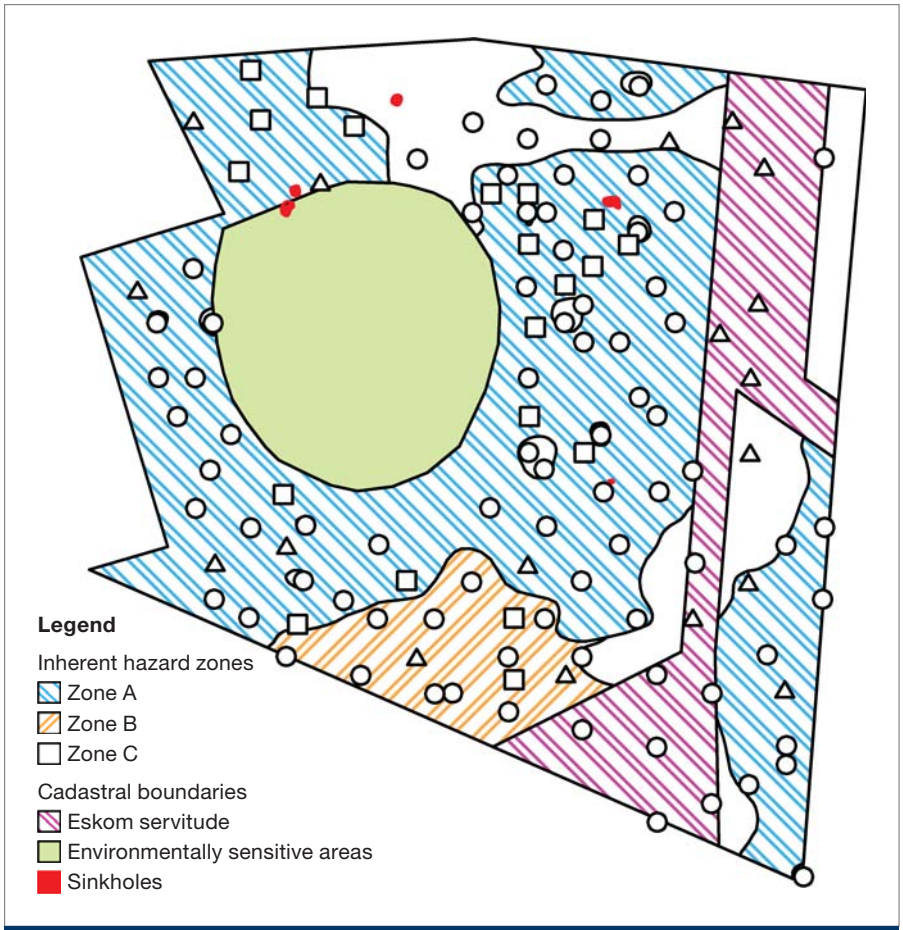

Figure 1: Hazard zone plan with borehole and sinkhole positions included

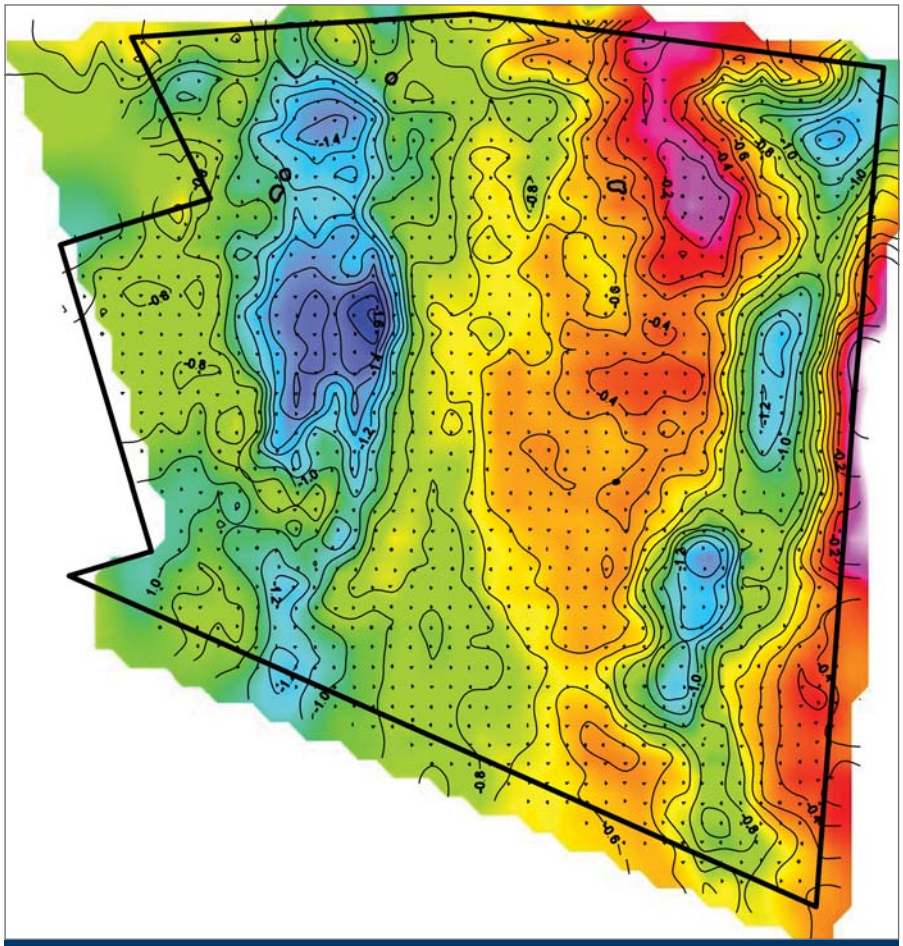

Figure 2: The 30 m gravity grid

some practical limit to the amount of detail that can be picked up in a gravity survey, and thus isolated pinnacles and very narrow or shallow solution features may not be detected. Drilling in dolomitic terrain is therefore always likely to find anomalous bedrock depths within what appears to be a uniform gravity feature.

THE CASE STUDY

A large area has been investigated for township development purposes in the eastern part of Centurion, Pretoria. The site is underlain by dolomite and chert of the Monte Christo Formation, Malmani Group, Transvaal Supergroup. An environmentally sensitive area was identified roughly in the centre of the site and this was excluded from the investigation. Sinkholes are known to have developed in the area and five small, old sinkholes were mapped within the bounds of the site.

Initially a 30 m gravity grid was used to cover the area, followed by percussion borehole drilling as is the norm. The initial results were not satisfactory and it was not possible to clearly demarcate areas with uniform hazard. Subsequent investigations resulted in a total of 118 boreholes being drilled under the guidance of three different consultants in an attempt to refine the hazard zones to a satisfactory level. Typical of many such investigations, the first attempt made use of the gravity survey and borehole results to produce a hazard zone map. The resulting plan contained large areas of shallow dolomite within which an unacceptably large number of boreholes indicated deeper bedrock. The subsequent investigations, involving only drilling, abandoned the gravity survey as a guideline, and attempts were made to create hazard zones by 'joining the dots', creating areas within which boreholes indicating deeper bedrock, with associated larger sinkhole predictions, seemed to be common. Zones such as these cut across the areas in which the gravity survey was predicting shallow bedrock. The end product (see Figure 1) ultimately made less sense than the first survey, and the regulatory authorities were not convinced that there was sufficient confidence in the results to allow development to proceed.

In an attempt to better understand the ground conditions and map a way forward, it was decided that detailed gravity surveys should be carried out across selected portions of the site. A 10 m grid

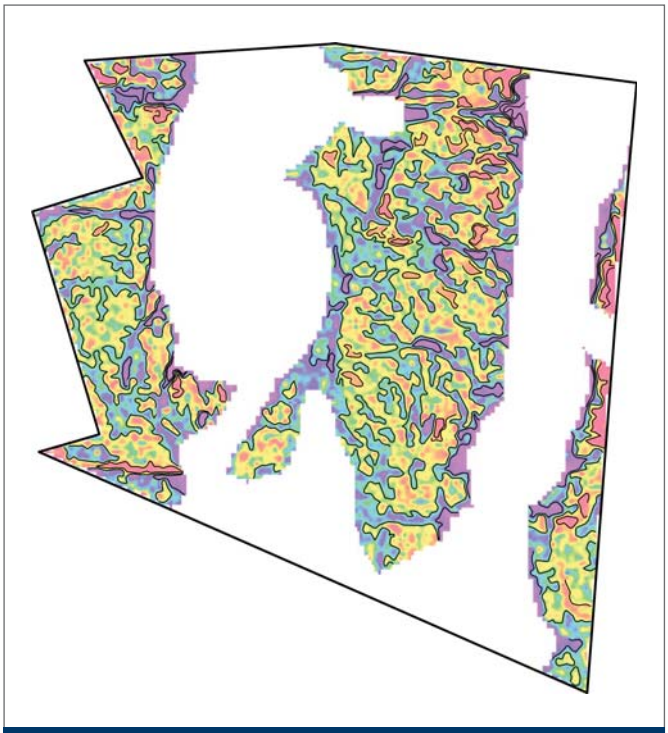

Figure 3: The 10 m gravity grid

spacing was used for the detailed gravity survey. This resulted in a significantly higher resolution of bedrock patterns than the previous survey conducted on a 30 m grid spacing. Not only did this higher resolution allow poor zones to be delineated, but it allowed better predictions to be made with respect to the width of solution features and the delineation of shallow bedrock.

The success of detecting 'grykes' or solution features is directly related to the spacing of the gravity grid. A feature smaller than the 'cell' size, which is half the spacing of the gravity grid, will not be detected. The detailed survey carried out on a 10 m grid for this investigation, therefore, allowed the successful delineation of all areas where a potential existed for medium or large-sized sinkholes to develop. A comparison of the difference in detail is given in Figures 2 and 3.

An additional 31 boreholes drilled were sited using the detailed gravity survey. These were often drilled across narrow zones of deeper bedrock predicted by the survey to confirm the width of solution features (grykes). This enabled new hazard zones to be identified with precision (Figure 4), and the site was divided into three zones as follows:

- **Zone A:** Zone A includes areas where shallow or outcropping bedrock is dominant. As the bedrock is considered to have cavernous conditions and overburden is not considered to be particularly competent, this portion of the site is considered to have a high potential for small sinkholes to develop and a moderate potential for the development of medium-sized sinkholes. It must be noted that a typical sinkhole size is not expected to exceed 3 m in diameter in these areas.
- **Zone B:** This zone includes areas where thick, more competent chert overburden overlies deeper dolomite bedrock. These areas are considered to present a moderate hazard level and it is likely that medium to large-sized sinkholes will develop in these areas, should they occur.
- **Zone C:** Zone C includes areas where bedrock is deep, solutions are wide, cavernous conditions exist and the overburden is not competent. These areas are typically associated with

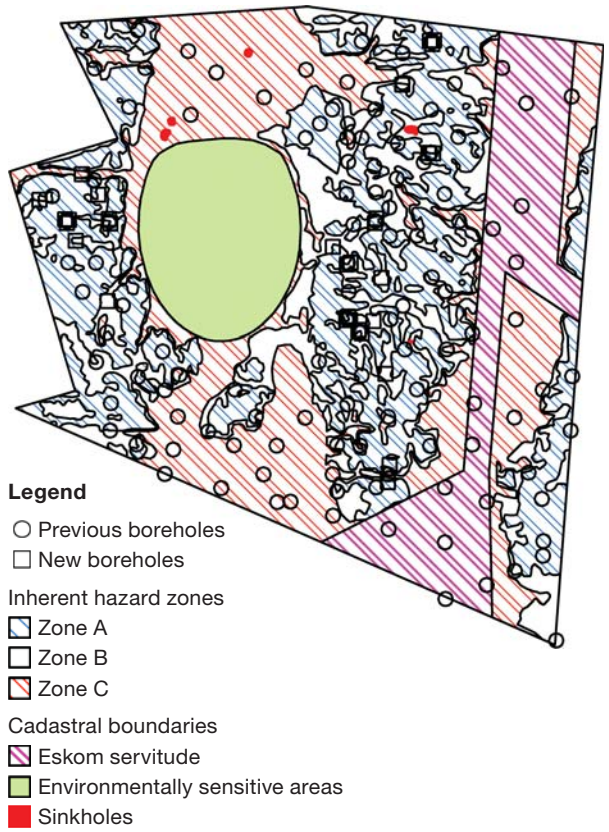

Figure 4: New hazard zoning

large gravity-low anomalies, and are considered to have a high potential for medium to large-sized sinkholes to develop. Isolated areas of shallow bedrock are present in Zone C, but these are considered to be too small to be of significance.

CONCLUSION

This case study demonstrates that far better resolution of bedrock topography is possible using a smaller gravity survey grid spacing. Appropriately spaced gravity grids allow for better zonation of sites, as well as better identification and prediction of the width of solution features within generally shallow bedrock areas. The additional cost of the gravity survey will be offset by reduced drilling requirements, and the production of a more confident zonation and development plan. In this specific case more land was deemed usable than the original investigations had indicated.

ACKNOWLEDGEMENT

The authors wish to gratefully acknowledge the interactions with and the contributions from Richard Day, geophysicist.

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Rigid inclusions – an innovative geotechnical solution for challenging ground conditions

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INTRODUCTION

Construction on soft clays has always posed a challenge for geotechnical engineers. Soft clay is characterised by low strength, stiffness and permeability, which lead to bearing capacity and long-term settlement-related problems if foundations are inadequately designed. Jones and Davies (1985), in their state-of-the-art-paper on soft clays, stated that the main challenge is to characterise the deposit, which is extremely variable both spatially and in its engineering properties. Due to this uncertainty, structures are generally founded on deep pile foundations which are often conservatively designed and expensive.

The Durban area has developed around the mouths of three main rivers – the Mgeni, Mbilo and Mlazi. The underlying estuarine deposits, locally known as the Harbour Beds (King & Maud 1964), are characterised by lenticular sand deposits intercalated with silts and clays overlain by thick layers of dark-grey, soft-silty clay (locally known as the Hippo Mud) which may extend to depths of up to 30 m.

A foundation solution was required for the 350 000 m^2 Clairwood Logistic Park development, located on the old Clairwood Race Course approximately 3 km northeast of the old Durban International Airport. The site is underlain by soft clays extending to depths of 35 m, and traditional piled foundations are financially not feasible. The proposed solution was therefore to opt for ground improvement with the use of *rigid inclusions* or *controlled stiffness columns* (CSC®).

Ground improvement with rigid inclusions requires concrete columns to be installed in a grid format and founded on rock or a competent soil layer. These concrete columns do not necessarily improve the mechanical properties of the clay, but reinforce the soil to create a composite soil/concrete mass with significantly improved mechanical properties. The system also requires a load transfer platform constructed above the column head to transfer load from the structure to the rigid inclusions, similar in function to a pile cap which transfers load from the column to the piles. The difference in the operating principle, in comparison to other foundation systems, is summarised in Figure 1.

HISTORY OF RIGID INCLUSIONS

Possibly the first application of rigid inclusion techniques for ground improvement was recorded in 1904 when engineers proposed to support the Mexican parliament building on driven metal inclusions not connected to the structure. Since then various case studies have been published, including Correa (1961) using piles inserted into a perforated hollow raft, Girault (1969) using overlapping piles in Mexico, Coles (1986) using driven inclined wooden piles with perforated planks for road foundations, Smoltczyk (1976) for road embankments supported by rigid inclusions topped by perforated caps in West Germany, and Gigan (1975) using vibro-driven micropiles to support a bridge abutment in France. By the late 1990s it was evident that rigid inclusion techniques have been and could be applied in various foundation solutions, but that a standardised approach was required for design and implementation techniques, as well as for inclusion material. In 1999 a proposal was made by the French Geotechnical Society for a national project on the topic. This ultimately

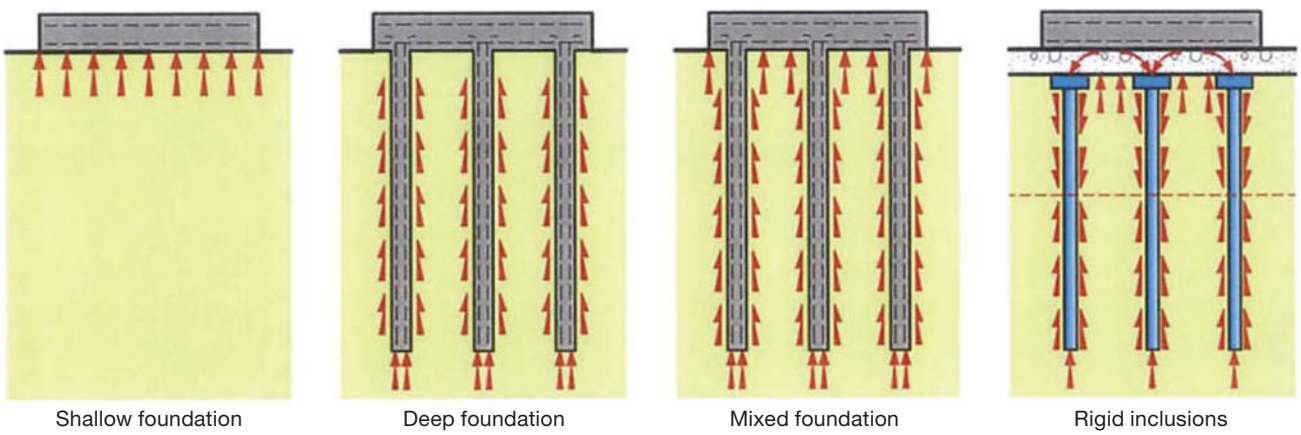

Figure 1: Operating principles of various foundation types (ASIRI 2013)

Figure 2: Liebherr LRB 255 crawler rig equipped with ring vibrators for driving steel tubes

led to a four-year national research project (ASIRI) in 2005, and the publication of *Recommendations for the design, construction and control of rigid inclusion ground improvement* in 2013 as a guideline for the design and implementation of rigid inclusions. The technique is now a well-established ground improvement solution used around the world.

INSTALLING RIGID INCLUSIONS

Franki Africa was awarded the ground improvement works for the Clairwood Logistics Park development in late 2016, which would include installation of over 45 000 rigid inclusions to depths of over 35 m. The work scope includes site characterisation, design, implementation, control and monitoring of the ground improvement system. Bigen Africa, assisted by SRK Consulting Engineers, provides independent review and control of the geotechnical works and the interface between the geotechnical and structural design.

The installation of rigid inclusions is being carried out by Liebherr LRB 255 crawler rigs specially equipped with model 32 VMR ring vibrators to drive temporary steel tubes to the required depths. In highly variable ground conditions, the use of the vibratory driving method allows rigid inclusion lengths to be installed based on the actual ground condition/profile rather than on a designed depth, which may be inadequate or over-conservative.

The rigid inclusions are being finished off with a gravel head (installed using Keller Vibrocats) and topped with 2–3 m engineered fill, which acts as the load transfer platform. The solution is schematically illustrated in Figure 3. The combination of rigid inclusions and gravel head, also known as a Hybrid Column (CMM®), reduces the risk of column head damage by construction vehicles or the environment, and reduces the punching stresses and moments on the floor slabs.

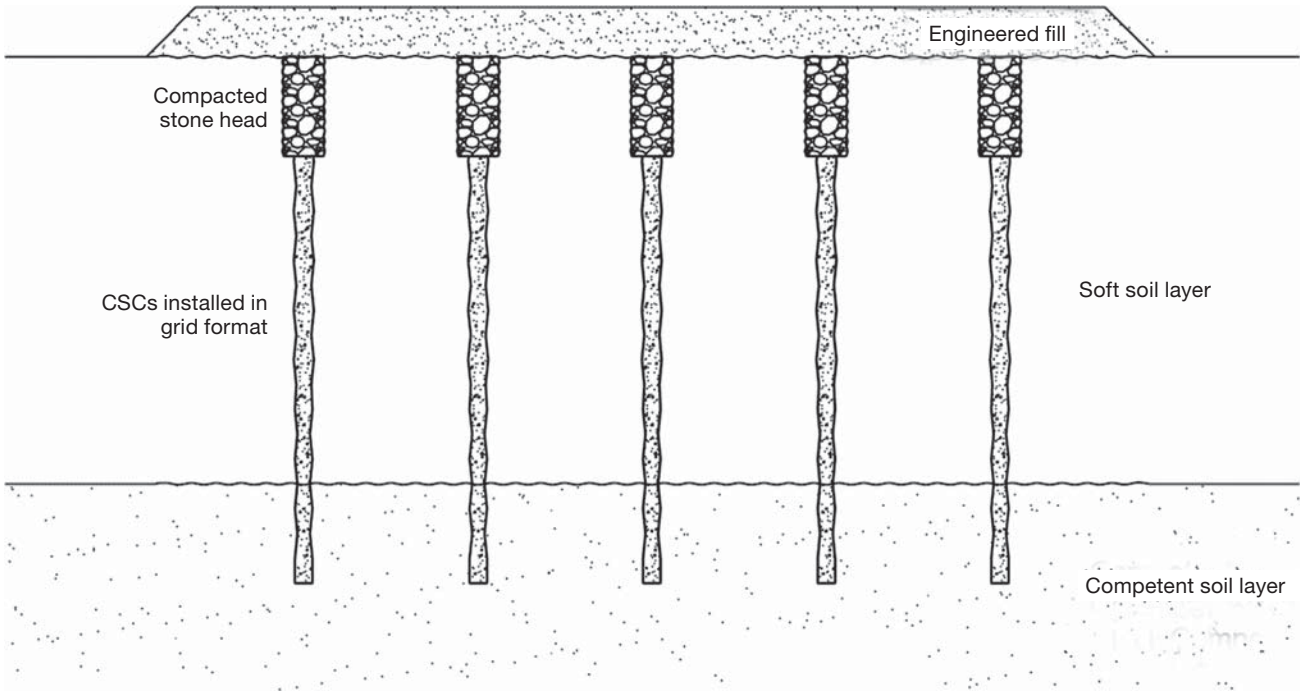

Figure 3: Illustration of soil reinforcement solution with rigid inclusions

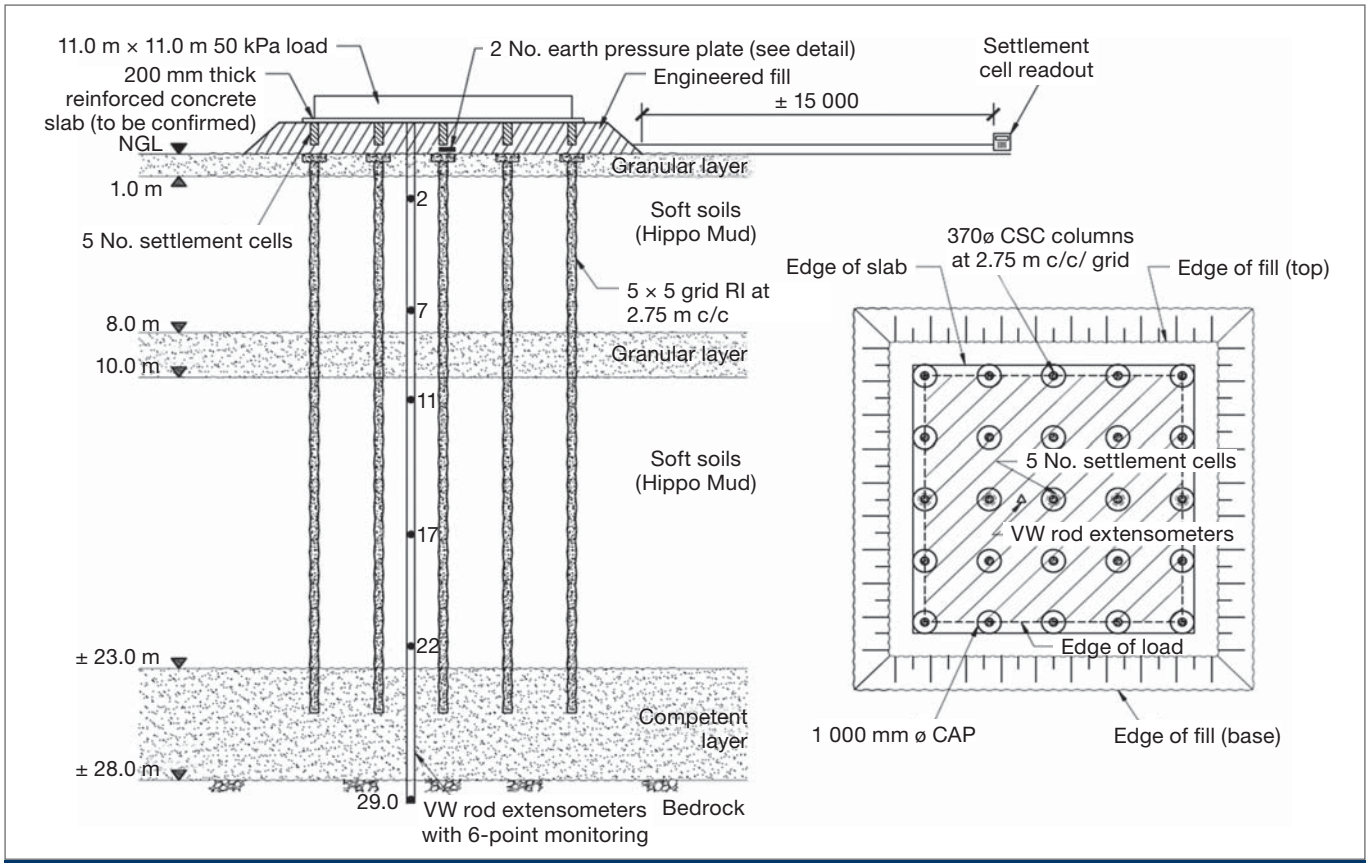

Figure 4: Layout and instrumentation of the full-scale field test

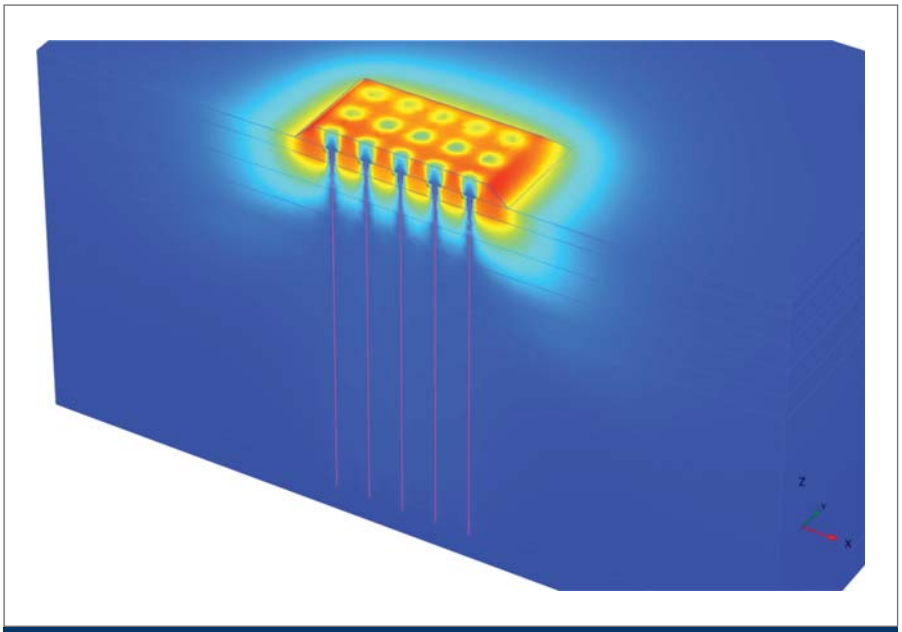

Figure 5: 3D finite element analysis of the soil displacement

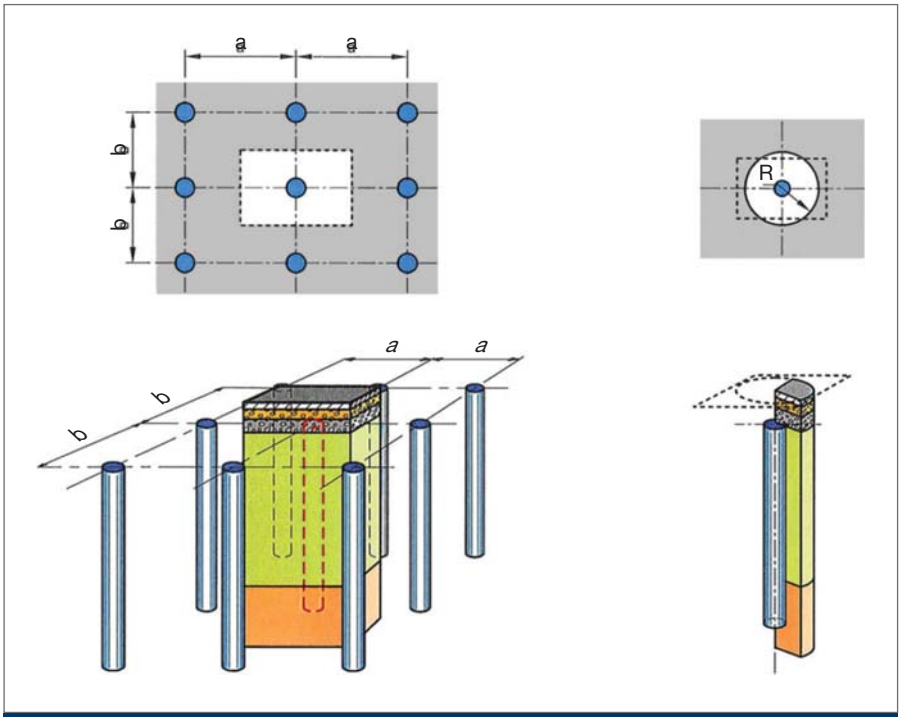

Figure 6: Axisymmetric model representing a single unit cell of distributed load supported by soil reinforced by rigid inclusions (ASIRI 2013)

As the rigid inclusion solution is untested in South African ground conditions, a full-scale field test programme was conducted before commencement of works to validate the viability of the system. The test programme included a grid of 25 rigid inclusions topped by 1.4 m thick compacted granular fill. The test pad was fully instrumented, with vibrating wire extensometers positioned at various depths to monitor vertical ground strains, hydraulic settlement cells and embedded survey points to monitor surface settlement, as well as pressure cells to monitor vertical pressure below the compacted fill. The tests not only confirmed the validity of the rigid inclusion system, but also provided information about which material properties could be back-calculated for design optimisation. In addition to the full-scale test pad, fully instrumented single column tests were also carried out as part of the pre-construction testing programme. Such tests are used to establish the load transfer characteristics of an inclusion in the ground, which are then used to 'calibrate' the finite element analysis.

Prediction and back-analysis of the test pad performance was carried out using 2D and 3D finite element analysis for both immediate and time-related settlement. Settlement under applied load was estimated at 35 mm, and compared well with the measured value of around 30 mm. However, time-related settlement behaviour was less comparable, with 90% consolidation estimated at between 8 and 12 months, and measured at just over 3 months. The reduced consolidation period, probably resulting from the lenticular nature of the alluvial deposit, greatly reduces the risk associated with time-related settlement, as any settlement resulting from the weight of the fill would occur before construction of the floors, reducing the differential settlement on the floor.

Rigid inclusions can be used to provide stability (ultimate limit state) or settlement control (serviceability limit state), and the latter was the requirement, paticularly for the warehouse floors. Without improvement, ground settlement is estimated at between 200 mm and 400 mm, with consolidation periods of between two and five years. The presence of rigid inclusions notably increases the vertical stiffness of the soil mass and reduces the stresses applied to the soft clays, thereby significantly reducing the settlement and consolidation periods.

The design of the ground improvement was carried out using both 2D and 3D finite element analysis. Underneath most of the warehouse structure, the conditions are one-dimensional and could be analysed using axi-symmetric finite element models as unit cells (shown in Figure 6). Oedometric Young's modulus was correlated to CPT results using correlations back-analysed from the pre-construction test programme. Axi-symmetric analysis results were compared to results from the load transfer method developed by Bohn (2015) as a sanity check. Axi-symmetric analyses require little computational time, and are used as quick checks for various ground conditions and fill scenarios. Axi-symmetric and 2D finite element models, however, are inadequate to assess building edge and corner effects, and partial floor loading conditions. Hence 3D finite element analyses were used in these cases.

Ground settlement is monitored using vibrating wire settlement sensors buried below the working platform level. The data from the reservoir is connected to a wireless logger box which sends the data to a site gateway powered by solar panels and contains a GSM modem. Data is sent

every 15 minutes to the Getec Database (a Keller company specialising in geotechnical instrumentation and monitoring solutions) and can be viewed via a web browser. The system provides realtime monitoring of the fill settlement, which can be used to validate the performance of the ground improvement works, and serves as an early warning system for potential inadequate design/works.

Figure 7 Aerial view of the ground improvement operations in the first area of the Clairwood Logistic Park development

CONCLUSION

Ground improvement with rigid inclusions has numerous advantages compared to conventional piled foundations, particularly in challenging ground conditions. As with all ground improvement techniques, structures are founded on inexpensive light/shallow foundations once the ground improvement has been completed. This generally leads to a significant reduction in the cost of the overall foundation system when compared with piled solutions which require pile caps, ground beams and thick rafts or slabs. Installation of rigid inclusions is significantly faster than conventional piling, particularly in challenging soil conditions, and often leads to programme and cost benefits for the project.

Furthermore, the inherent redundancy in ground improvement solutions provides reduced risk in challenging ground conditions (in ground characterisation, design and implementation) when compared to piled foundations, which provide the full bearing resistance for the structure. It is an alternative to piling for structures over large footprints with distributed loads, such as warehouses, storage reservoirs, treatment plants, basins and retention facilities, road embankments, etc, which often have stringent differential settlement criteria. It is, however, not suitable for structures with highly concentrated loads, or structures with stringent total settlement requirements.

REFERENCES

The list of references is available from the author. ◻

The use of CSW testing to estimate bedrock depth

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One of the main criteria of the project was that the production of the existing plant should not be affected by the construction of the expansion project.

INTRODUCTION

This article discusses the use of the Continuous Surface Wave (CSW) testing method to estimate the depth of bedrock for an expansion project for the Grootegeluk mine near Lephalale (previously Ellisras) in the Limpopo Province. CSW testing is part of the family of geophysical test methods. Over the past 30 years the use of geophysical test methods as part of geotechnical site investigations has increased steadily (Stokoe *et al* 2004), and CSW testing is part of the seismic wave testing methods. The test is a seismic technique for the determination of ground stiffness by measuring the velocity of seismic wave propagation along the ground (Matthews *et al* 1996; Stokoe *et al* 2004).

Seismic test measurements have a range of different applications, such as the following:

- Classifying ground
- Estimating engineering parameters such as stiffness and Poisson's ratio
- Calculating settlement for 'static' foundations
- Estimating design parameters for machine foundations
- Investigating liquefaction potential
- Judging the rippability of in-situ soil.

In South Africa a range of different seismic field tests are available, such as CSW, seismic cone test, down-hole test,

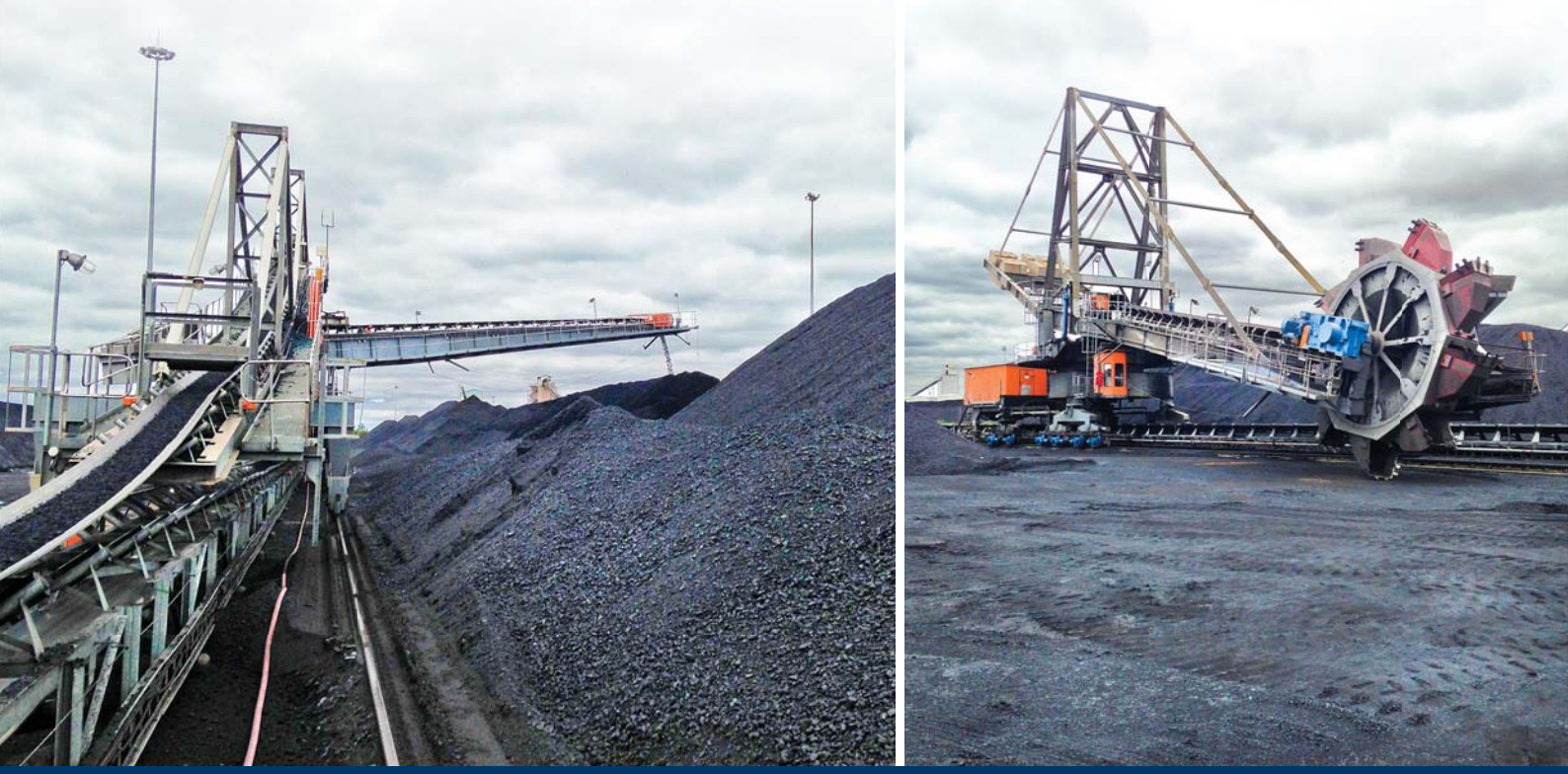

Figure 1: Existing GG6 stacker (left) and bucket wheel reclaimer (right)

cross-hole test and seismic refraction test. This article will focus on the utilisation of seismic testing as a tool for ground classification.

PROJECT OVERVIEW

The GG6 Expansion project aims to increase semi-soft coking coal production at the Grootegeluk (GG) coal mine through modifications and additions to the existing GG2/6 plant and associated materials handling systems. The project called for the construction of a new 6 000 t coal silo, coal beneficiation plant, upgrading and expansion of the existing GG6 stockyard, and numerous overland conveyors connecting the beneficiation plant with the stockyard.

One of the main criteria of the project was that the production of the existing plant should not be affected by the construction of the expansion project. This was especially challenging in the stockyard area, as the extension of the rail beams for the stacker and bucket wheel reclaimer (see Figure 1) had to be constructed in close proximity to the existing stockyard feed conveyor. Deep excavations would be challenging, and it was important for the design engineers to have a good understanding of the depth of the underlying bedrock.

Figure 2: Typical layout of a CSW test

SITE GEOLOGY AND GROUND CONDITIONS

Based on the review of geological map 2326 (Ellisras), it was evident that the proposed GG6 plant site is entirely underlain by basaltic bedrock of the Letaba Formation of the Karoo Supergroup. Over the greater part of the Ellisras Basin these lava flows have been eroded away over time, exposing the underlying Karoo sedimentary strata, which makes this site a challenge with unknown founding depths of the undulating sub-surface basaltic bedrock formation. The entire area is covered by wind-blown (also referred to as aeolian) sands.

The Kalahari aeolian sand found over the study area is generally known for its high collapse settlement potential. This phenomenon is caused by a relatively high strength at natural moisture, which decreases rapidly when moisture is added. When this occurs under load imposed by structures, sudden collapse settlement is often the result, thus potentially leading to serious damage to any infrastructure founded on this material.

Table 1: Typical shear wave velocities (Borcherdt 1994)

Material type	Shear wave velocity (Vs)
Hard rocks	Vs > 1 400 m/s
Firm to hard rocks	700 m/s < Vs < 1 400 m/s
Gravelly soils and soft rocks	375 m/s < Vs < 700 m/s
Stiff clays and sandy soils	200 m/s < Vs < 375 m/s
Soft soils	100 m/s < Vs < 200 m/s
Very soft soils	50 m/s < Vs < 100 m/s

An intrusive geotechnical study was carried out at the inception of the project and, based on excavated test pits and available literature, it is evident that the shallow near-surface soil conditions across the site are generally homogenous. The limited testing carried out during the planning stage triggered the need for a less evasive method of determining the founding depths of the proposed structures. Coupled with the complexity of maintaining operations (production), lock-out of the study areas was non-negotiable, and this resulted in further investigations with reference to less intrusive methods of determining founding depths.

SEISMIC TEST AS A TOOL FOR GROUND CLASSIFICATION

During seismic testing geophones are used to measure the wave speed of mechanically generated waves that travel through the ground. The waves are generated by a range of different methods, either by hitting the ground with a hammer or by

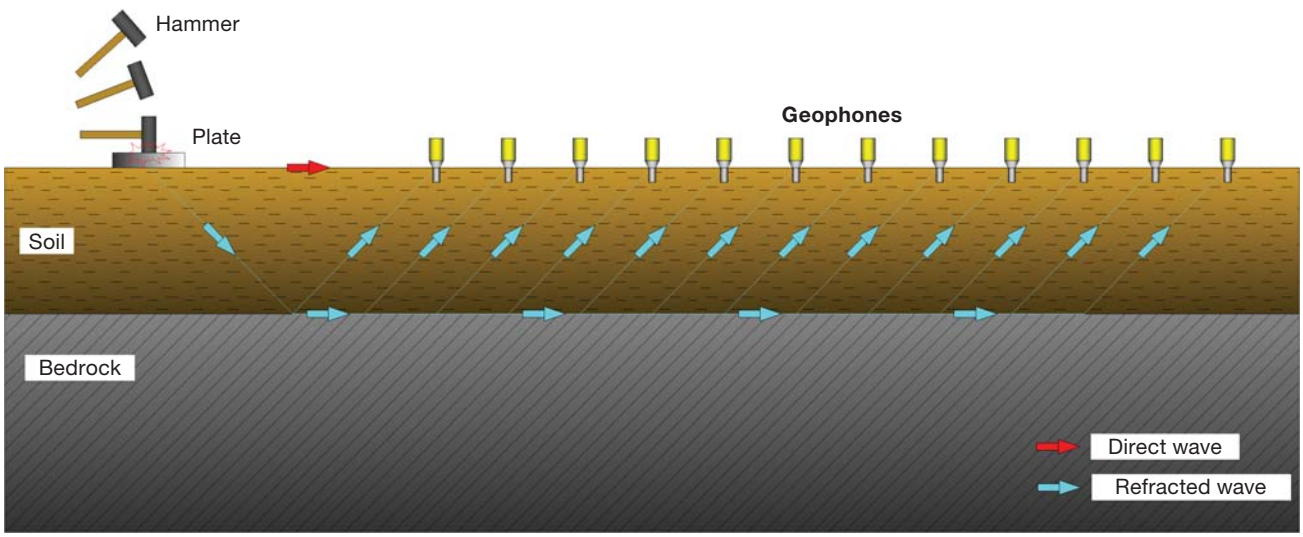

Figure 3: Seismic refraction test setup

a vibrating shaker. The velocity of a mechanically generated shear wave is affected by the medium that it travels through. The denser the medium, the higher the velocity. Thus, by measuring the shear wave velocity of any ground stratum, a ground classification can be established based on the measured shear wave velocity. These typical velocities can be seen in Table 1.

For the investigation of the GG6 project, only two surface wave test methods were considered – the CSW test and the seismic refraction test. With the CSW test a mechanical shaker is placed on the ground together with 3–5 geophones (see Figure 2). The shaker is used to generate Rayleigh waves with a range of different frequencies (low frequencies have a deeper penetration). Depending on the size of the shaker, waves can penetrate up to twenty metres deep. The output produced from the test gives the shear wave velocity profile with depth, which can then be used to classify the ground profile.

With the seismic refraction test pulses of low frequency seismic energy are emitted by a source such as a hammer blow to the ground (refer to Figure 3). The seismic waves propagate downward through the ground until they are reflected off the subsurface, such as bedrock. The refracted waves are detected

Soil legend	Depth (m)	Description
	2.1	**Aeolian sand** Silty fine sand
	3.37	**Coarse sandy gravel** Completely weathered basalt boulders becoming highly weathered soft rock
	4.37	**Aeolian sand** Silty medium, to coarse sand
	5.37	**Basalt** Completely weathered very soft rock
	7	**Basalt** Highly to completely weathered very soft to soft rock

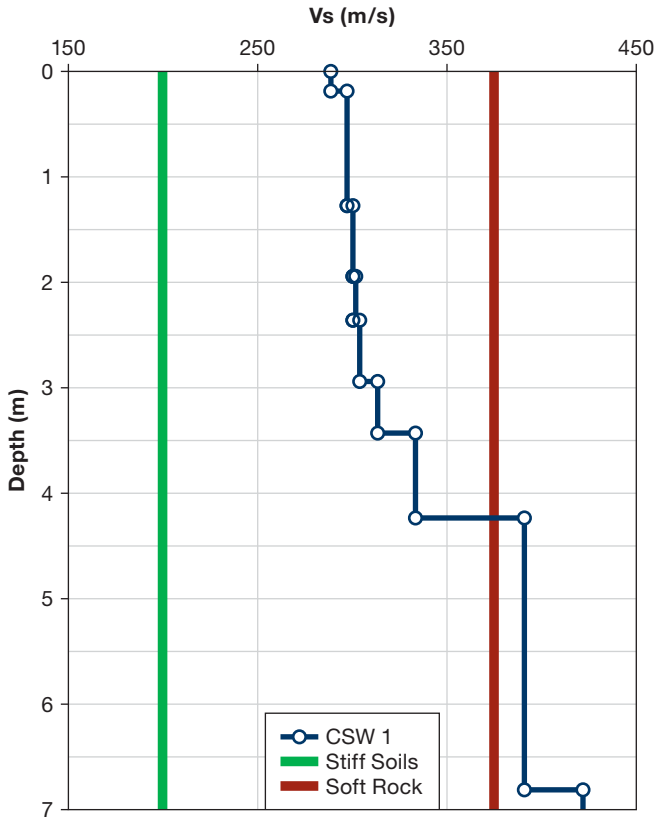

Figure 4: Borehole profile log (left) versus CSW test results (right)

by arrays of 24 to 48 geophones spaced at intervals of 1–10 m. This test is commonly used to estimate the depth to bedrock.

Under normal circumstances the seismic refraction test would have been the preferred test method to estimate the depth of the basalt layer for the project site, as its depth measurement is more accurate than that of the CSW. But the project site was located in close proximity of numerous conveyors, crushers and other sources of vibrations, and the seismic refraction test is very sensitive to background noise, due to the fact that there is no control over the frequencies of the waves produced. It was therefore decided to rather use the CSW test method to determine the depth of the basalt layer across the project area, even though the depth measurements of the CSW test is less accurate than that of the seismic refraction test. Another deciding factor was the speed at which the CSW test can be conducted – the CSW test only requires five geophones versus the 24 geophones of the seismic refraction test. With the project site being located on a mine, a limited time window was available for the tests to be conducted, as portions of the plant had to be shut down during testing in order to limit background noise.

CSW TESTING AND RESULTS

In June 2016 BVi appointed CSW Soil Engineering (Pty) Ltd to conduct 17 CSW tests across the project site. It was decided to also do one scan at the same location of a borehole that had been drilled and profiled during a previous investigation, in order to be able to compare the results from the CSW test with those of a profiled borehole to ensure that the results correlated. The CSW test results at the borehole indeed correlated sufficiently with the profile log of the borehole, as can be seen in Figure 4.

It can be seen from Table 1 that the lower boundary for stiff soils is 200 m/s (green line in Figure 4) and the lower boundary for soft rock is 375 m/s (red line in Figure 4). In the figure, the borehole log indicates that very soft basalt bedrock is encountered at 4.37 m. The CSW profile shows that the shear wave velocity increases to a speed above 375 m/s at ±4.3 m deep. These results are in line with the values listed in Table 1. For the analysis a target shear wave velocity of at least 375 m/s was used to identify the point where the ground profile transitioned into basalt bedrock.

Figure 5: Testing along the proposed positions of the anaconda conveyor (left) and the new stacker rail beams (right)

Testing was mainly conducted in the new stockyard area. At a length of 800 m, the depth to bedrock could severely impact the cost of founding the rail beams for both the stacker and the reclaimer. Testing was conducted in a straight line at ±100 m intervals (refer to Figure 5 for a layout of the CSW testing). CSW testing was also conducted at critical foundations along the new stockyard feed conveyer (nicknamed the 'anaconda conveyor' due to its roller coaster-like profile).

The CSW test results are shown in Figures 6–8. It can be seen from all three graphs that the depth at which a shear wave velocity of 375 m/s is reached varies from 2 m to beyond 7 m. These results, together with the fact that construction of the new stacker rail beams would have to occur ±2 m away from an existing conveyor which has to stay operational during construction, necessitated the design engineers to look for alternative methods of founding the rail beams. With the assistance of Dr Nicol Chang from Franki Africa it was decided to found the rail

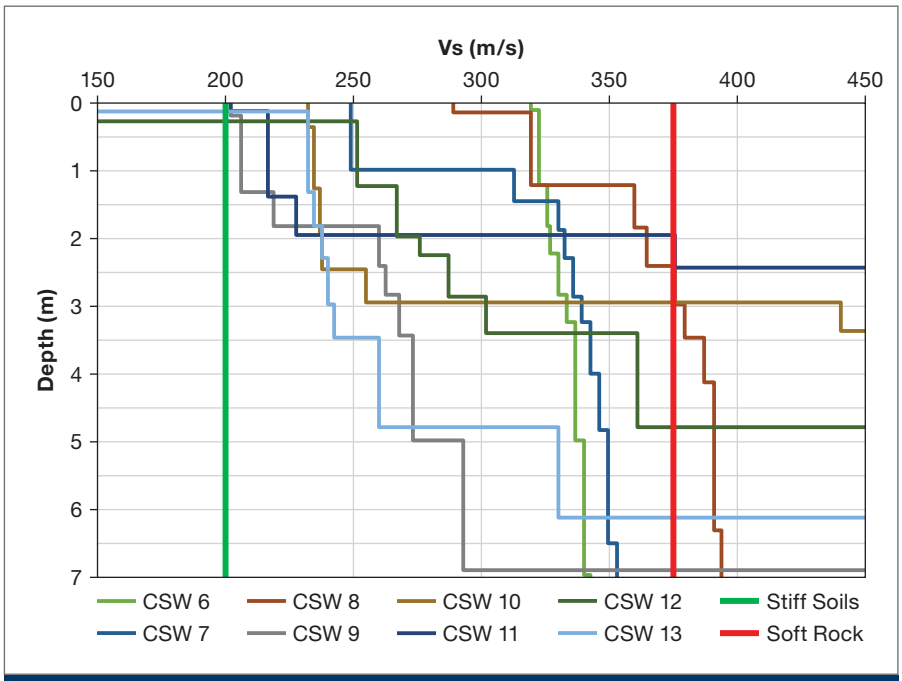

Figure 6: CSW results along the stacker rail beams

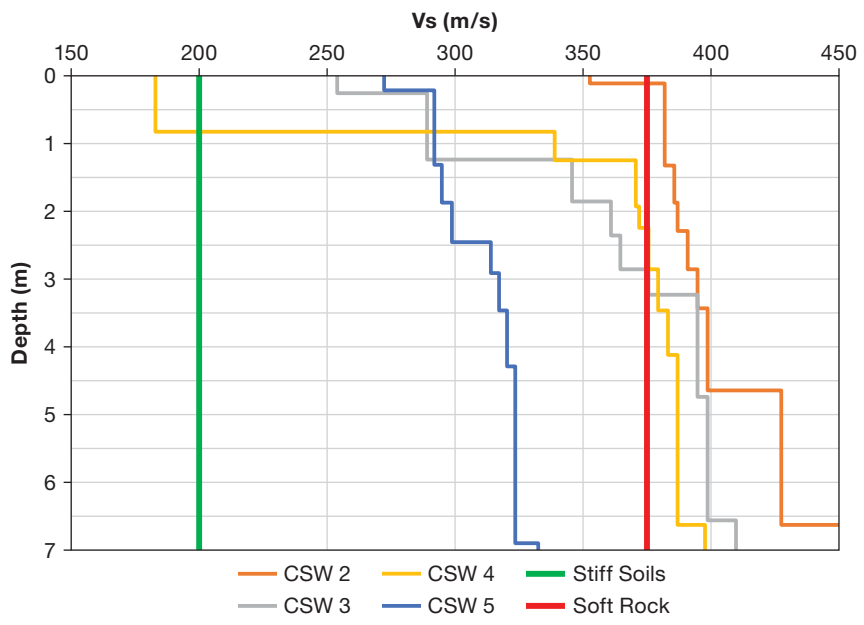

Figure 7: CSW results along the reclaimer rail beams

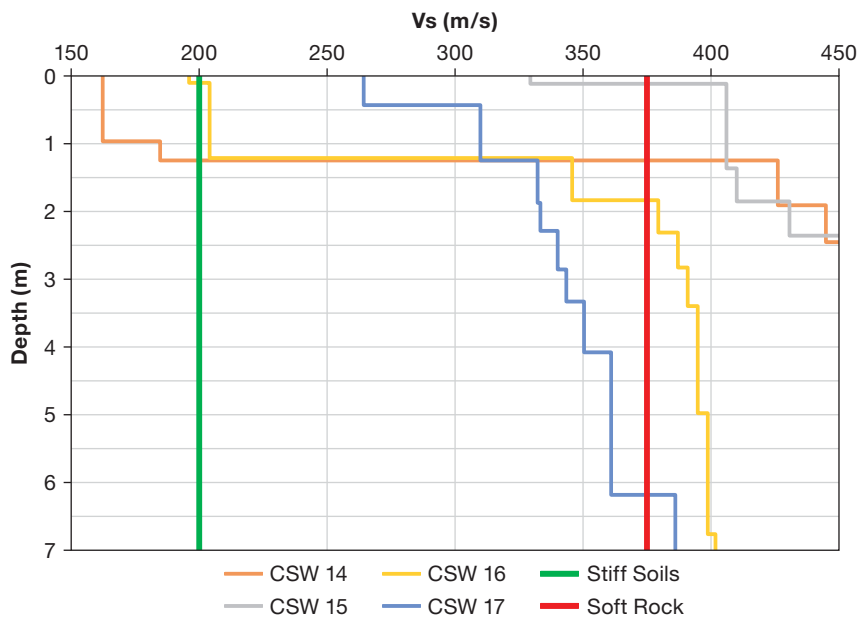

Figure 8: CSW results along anaconda conveyor

PROJECT DATA	
Client	Exxaro
Principal consultants (materials handling)	LSL Consulting
Sub-consultant (civils)	BVi Consulting Engineers
Sub-consultant (process)	JHDA
Construction period	June 2017 – June 2020

beams of both the stacker and reclaimer on 600 mm diameter auger piles, instead of excavating and replacing the aeolian sand with competent material. The pile solution was deemed to be more cost and time-effective. The auger rigs will drill down until it refuses on the weathered basalt.

CONCLUSIONS

The CSW testing conducted for the GG6 project proved to be an effective method to gain a better understanding of the soil profile below a large area. Seventeen tests were conducted over a two-and-a-half-day period. A possible alternative that could match the production speed would have been a DPSH test. In the case of the GG6 project the DPSH test was not considered an option, due to the fact that a probe would have to be driven down into the ground and would require an excavation permit from the mine in order to ensure that there are no electrical cables or any other services that the probe might damage. As the CSW test is a non-destructive test, no excavation permit was required and the testing could be done in the three-day window that was allowed by the mine for short duration non-risk work. For any period longer that three days, or for work that is deemed to carry a risk of injury (such as test pits and DPSH probing), a full medical and induction to the mine would have been required, thus substantially increasing the cost.

The results from the CSW test also gave the design engineers a better understanding of the soil profile below the largest and most critical (in terms of risk associated with founding conditions) portions of the project site. The results allowed the design engineers to sufficiently identify risk in terms of founding conditions and enabled them to choose the correct method of founding the relevant structures. It also allowed the cost of founding works to be estimated more accurately than what would have been possible without the CSW results.

The CSW test proved to be a handy tool to estimate the approximate bedrock depth for design purposes. Caution should, however, be taken in areas with sand that has a collapse potential. Due to the cementation of the sand, it has a very high stiffness, but this stiffness is lost as soon as the sand gets wet and a load is applied. It is advised that the CSW test should be used in conjunction with conventional tests pits to gain a better understanding of the in-situ soil.

ACKNOWLEDGEMENTS AND REFERENCES

The authors would like to thank Professor Gerhard Heymann (University of Pretoria) and Dr Nicol Chang (Franki Africa) for their assistance during the execution of this project.

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The list of references, as well as a list of the works cited, is available from the authors. ◻

Geotechnical research at WITS

INTRODUCTION

The geotechnical group of the Civil Engineering Department at the University of the Witwatersrand (WITS) has undergone exciting developments in the last few months. One of the academics of the group, Dr Luis Torres-Cruz, completed his PhD last December, and the group will soon be adding a new member, Dr Thushan Ekneligoda, who obtained his PhD at the Royal Institute of Technology in Sweden, and previously worked as a research fellow at Nottingham University in the UK. Dr Ekneligoda has a keen interest in numerical modelling and underground coal gasification. In the midst of these developments the group continues to pursue its research agenda, of which we provide some highlights below.

INSIGHTS FROM INDEX PROPERTIES

The importance of soil index properties has long been recognised in geotechnical engineering. Classic texts, such as Terzaghi and Peck (1948), highlight the practical importance of establishing approximate correlations between index properties which are easy to measure (e.g. fines content, limit void ratios, Atterberg limits) and mechanical parameters whose direct measurement demands more time and resources (e.g. steady state line, friction angle, compressibility). The development of these correlations is the subject of intense research worldwide, and at WITS we are contributing to this ongoing conversation. For example, findings from Torres-Cruz (2016) indicate that the vertical position of the steady state line of non-plastic soils can be correlated to the minimum void ratio, and that this correlation is independent of particle angularity and of the particle size distribution of the soils. These findings challenge the view that the steady state line can be explained in terms of the fines content, as has been suggested by other authors (e.g. Rahman & Lo 2008).

Similarly, efforts are also being made to explore the correlation between the Atterberg limits and the mechanical behaviour of non-plastic soils. The starting point is the fact that the definitions of the liquid and plastic limits, commonly used for soil classification purposes, are largely arbitrary. Accordingly one can expect that the strength with which these limits correlate to mechanical parameters will

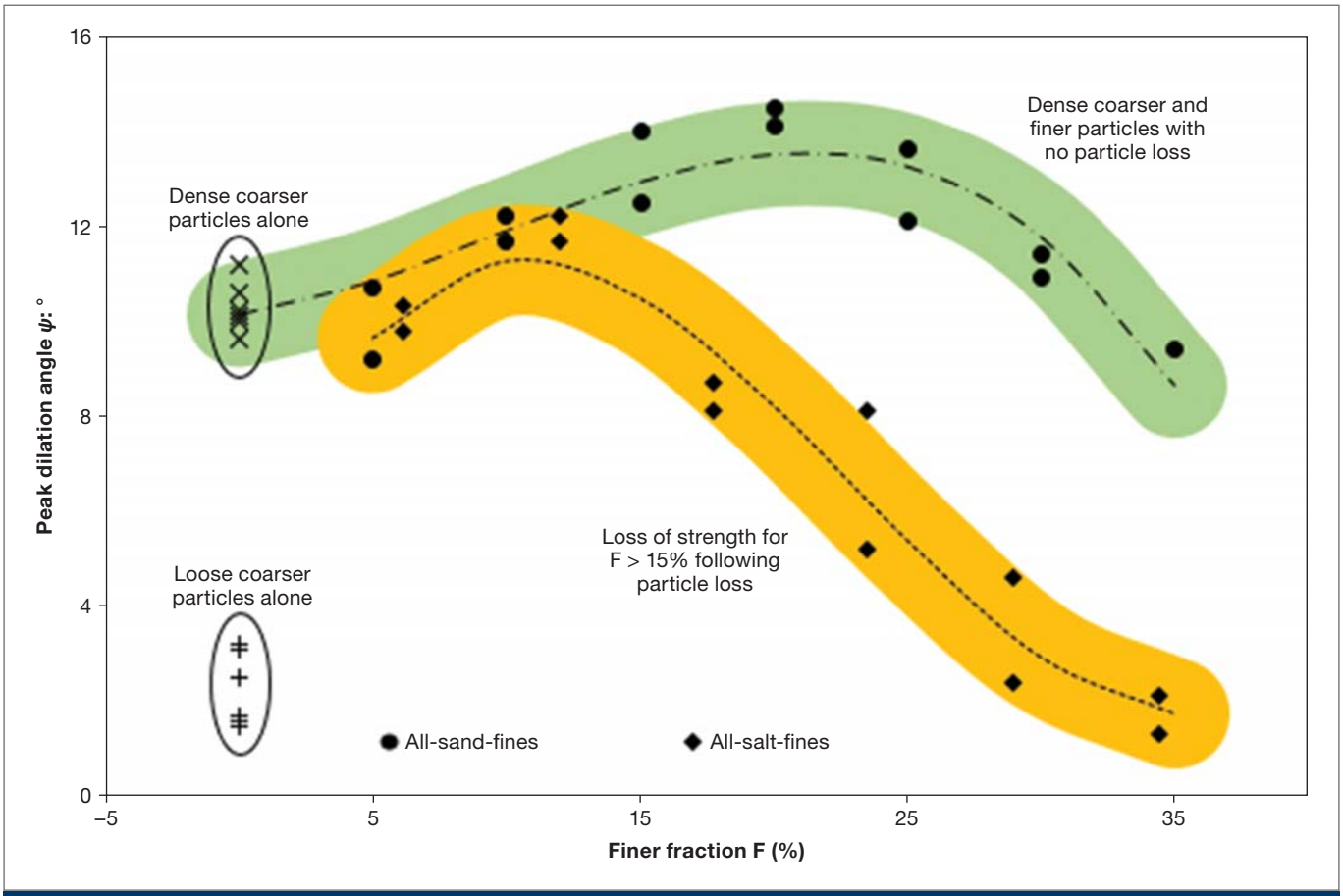


Figure 1: Typical results from variedSB tests investigating internal erosion

be compromised due to their arbitrary definitions. We are thus tackling the question: Is it possible to interpret a soil's consistency limits in a less arbitrary manner, and consequently achieve stronger correlations with mechanical parameters? Time (mostly spent in the laboratory!) will tell.

INTERNAL EROSION

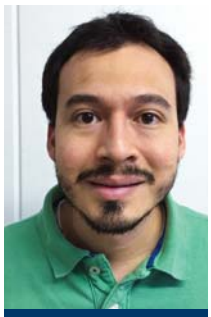
Particle loss from internally unstable soils can result in distress (termed suffosion) in some cases and in other cases no distress (termed suffusion). Recent research at WITS, using salt dissolution as an analogue for the eroded particles, has explored why this may be the case, using an in-house apparatus called the vertical axis restrained internal erosion direct shear box (variedSB) (MacRobert *et al* 2015; MacRobert & Day 2016). This showed that internally unstable soils can have two different fabrics, depending on the percentage of erodible particles. When the percentage of erodible particles is lower than a transition finer fraction ($F_t \approx 15\%$), the loss of finer particles has a negligible effect on shear strength (see Figure 1). This behaviour is due to one type of fabric in which coarser particles dominate inter-granular load transfer. However, when the percentage of erodible particles exceeds this transition value, the loss of finer particles results in a reduction in shear strength, which becomes greater as the finer fraction increases. This behaviour is due to a second fabric in which finer particles increasingly hinder coarser particles from attaining a dense arrangement. In this second fabric, coarser particles also dominate inter-granular load transfer so that they are left in a very loose arrangement following finer particle loss.

UNSATURATED SOIL MECHANICS

The Geoffrey Blight Soils Laboratory at WITS recently purchased a HYPROP

Figure 2: HYPROP device for determining soil moisture characteristics

Figure 3: Dr Irvin Luker handling apparatus to be used on an 8 500 kg drop-mass for the 'rapid' load capacity test



Dr Luis Torres-Cruz

Charles MacRobert

device – a device that uses the Wind/Schindler evaporation method (Wind 1966; Schindler 1980) to determine soil water characteristic curves and unsaturated hydraulic conductivity functions. The device utilises two precision mini-tensiometers to track developing suctions as a 250 ml soil specimen dries out on a laboratory scale. The device automatically generates the moisture characteristics, following initial specimen setup in shorter periods, than more traditional methods (Decagon 2017). The purchase of this device, along with various in-situ soil moisture and suction sensors, is aimed at developing methods to make unsaturated soil mechanics more accessible to local engineers.

TESTING OF FOUNDATION PILES

Dr Irvin Luker is now fully occupied at WITS in researching and developing techniques that are new to South Africa for testing foundation piles. These include the 'rapid' method of measuring the load-carrying capacity of any type of pile, integrity testing of the concrete in cast-in-situ piles, and a new type of strain rod for measuring the longitudinal strain in piles during a load test. The latter technique shows in detail the way in which load is transferred from any pile into the ground.

Figure 3 shows apparatus to test a mechanism to be used on an 8 500 kg drop-mass for the 'rapid' load capacity test.

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Soil Mechanics Research Group at CUT

WHY THE RESEARCH GROUP?

The problem of founding structures on expansive soils began receiving considerable attention in South Africa in the late 1950s. The major mining houses were then experiencing heave problems in accommodation provided for their workforce, particularly at their mines in the Free State, and they then provided funding for geotechnical researchers to investigate the problem. Some of the best soil mechanics experts in the country, including Professors JE Jennings and K Knight, were involved in this research. However, funds for such investigations have subsequently largely dried up, hence little research has been done on this problem since the 1980s.

The question of expansive soils has again come to the fore in South Africa since the new political dispensation came into being in 1994. A rapidly emerging black middle class, with aspirations of a home of their own, led to burgeoning construction of single-storey houses. Attempts by the government to provide low-cost housing for the poorest section of the community also led to the construction of millions of small, light houses. However, large numbers of failures have become common and many of these houses had to be demolished and replaced. In many cases, however, the replacements seem to have little prospect of a significantly longer useful life than the ones they are replacing.

Communication with Professor Derek Sparks (an expert in expansive clays at the University of Cape Town), led to the realisation that soil testing was needed. Literature searches, however, indicated that satisfactory answers were not readily available. The Department of Civil Engineering at the Central University of Technology, Free State (CUT), was approached in 2010 for possible involvement through its soil mechanics laboratory, and in 2011 the Soil Mechanics Research Group (SMRG) was formed to examine problems around the foundations of light structures on expansive clays. The late Professor Geoff Blight from the University of the Witwatersrand kindly agreed at the time to act as advisor. This article introduces the research group, shows what progress has been made towards understanding why current procedures appear to be inadequate, and offers possible solutions to these problems.

EXAMPLES OF FAILURE DUE TO HEAVING CLAY

In 2013 members of the SMRG visited Kimberley to examine three housing developments. A number of points of concern were noted at two sites, which were already at an advanced stage of construction. Soil samples were taken from these sites, as well as from a proposed development of 114 rental sites. Analysis of these samples suggested that the currently used test methods are inadequate to assess these soils, and that the test sites could expect expansion problems despite a favourable geotechnical report. Significant heave damage has indeed been reported – Photo 1 shows an example of typical damage observed at several of these sites.

Photo 1: Example of a house which became unfit for habitation before receiving its first coat of paint

Photo 2: Testing by fall cone conducted by research student, Zandri, and research assistant, Charlotte

Photo 3: Post-graduate student, Priscilla Monye, showing the settlement container that was designed to be waterproof while having one side removable for the extraction of settled samples of sand, silt and clay

INEFFECTIVENESS OF PRESENT TESTS

Foundation design for most light structures in South Africa, and in particular for low-cost housing, relies heavily on particle-size analysis and the determination of Atterberg limits. The tests for these properties were performed in commercial materials testing laboratories at the time, using the procedures of the CSIR's Technical Methods for Highways Part 1 (TMH1). SANS 3001 has now been phased in to replace TMH1. Both are primarily concerned with road construction, and investigations done by the SMRG indicated serious shortcomings in both of these norms in the context of foundation design for light structures. A paper titled "Shortcomings in the standard procedures for assessing heaving clays in foundation design" was published in 2015 in the *Journal of the South African Institution of Civil Engineering* [57(2): 36–44]. In this paper it was indicated that the liquid limit (LL), in particular, appears to require urgent attention.

SHORTCOMINGS IN THE ESTIMATION OF THE LIQUID LIMIT WITH CASAGRANDE CUP

Most engineers rely on values of LL determined by commercial laboratories. As commercial laboratories strive to make their services competitive and affordable to their clients, they perform a one-point procedure. Hence they take the specified mixing time of ten minutes to mean that the total time, from first adding water to the oven-dried sample until transfer of the mixed material to the Casagrande Cup for immediate testing, is to be exactly ten minutes. A team of six testers from the SMRG measured liquid limit and plasticity index by the one-point and by the flow-curve methods. Test results were compared with those from an accredited commercial laboratory. For clays with low heave potential there was little difference. As the activity of the clay increased, however, so did the discrepancies. In the case

of very active clay, the discrepancy in liquid limit is severe (one particular case, LL 50 vs 71, showed a difference of 42%) The discrepancy in plasticity index (PI) is even greater (in the above case the PI was 24 vs 44, a difference of 83%). Values for plastic limit (PL) were in good agreement in all cases, suggesting that the discrepancy in the PI was due to the LL only. Results from these tests were used to predict heave using several published methods. In the case of very active clays, heave predictions based on the flow curve results could be more than double those based on the one-point method.

LIQUID LIMIT USING THE FALL CONE

The fall cone has become the standard method of determining liquid limits in many countries, as it is considered to be less prone to operator error, more consistent in its results and generally more reliable than the Casagrande Cup. There also seems to be less scope for operator error than with the Casagrande apparatus. The only immediately obvious operator input prone to error appears to be adjusting the point of the cone until it just touches the surface of the sample. The Casagrande test on the other hand has a number of causes for concern which are raised repeatedly – the question of operator judgement as to when the specified length of the groove has closed, the problem of keeping a regular timing of two taps per second, the need for occasional adjustment of the distance of fall of the cup, the question of the hardness of the base on which the apparatus stands which affects the severity of each blow, etc. However, one not immediately apparent disadvantage of the fall cone is the large size of the sample required for the test – about twice as much soil is required as for the Casagrande test. Another not so obvious problem is the care needed when filling the sample mould. If air is trapped in the angle between the base and the walls, the resistance to the fall of the cone is significantly reduced as the entrapped air compresses.

The SMRG's evaluation of the fall cone is being done using multiple testing for each soil, and although the tests have been on-going for many months, it will still be quite some time before enough samples have been tested to meaningfully compare the old (cup) with the new (cone). Furthermore, an attempt to address other objections to the use of the fall cone is being conducted at the same time. The possibility of reducing sample size to something similar to that of the Casagrande apparatus, simplifying sample preparation, and reducing or eliminating the problem of air entrapment is being investigated. There is also the possibility of making the test more attractive to all concerned by obtaining the PL from the results of the same test. If the PI can be obtained from one procedure, the rolling of threads could be dispensed with. That procedure is even more severely criticised as being operator dependent (and hence potentially more unreliable than the Casagrande LL procedure). If this aspect of the investigation proves successful, it could make the fall cone a very attractive alternative (Photo 2 refers).

SHORTCOMINGS IN ESTIMATION OF PARTICLE-SIZE DISTRIBUTION

Suction tests on samples taken from the housing development mentioned previously showed considerable heave potential in spite of a hydrometer analysis showing only 6% clay fraction.

A microscopic investigation was undertaken in an attempt to understand why this should be. Clay is not normally considered suitable for analysis by light microscope. Some of the reasons for this are that dried clay forms dense agglomerations of particles which are difficult to differentiate, in suspension Brownian motion makes clay particles of about 1 µm or less very difficult to observe, high-powered lenses have only a small depth of focus, and it is difficult to distinguish between very small silt particles and clay particles. Also, Scanning Electron Microscope (SEM) images of very high magnification, great depth of focus, and clarity of a high order are available and appear to give an excellent representation of clay particles. However, preparation for SEM images involves such treatment as drying and gold-plating. The hydrated clay particles in the hydrometer are considerably different to dry particles. Also, labelling the clay particles with methylene blue makes it possible to recognise clay minerals and gain an insight into their behaviour in the hydrometer. Photo 4 shows a typical view of a soil suspension following chemical dispersion and high-speed mechanical stirring, as specified in SANS 3001 GR3. The large agglomeration of silt and clay-sized particles appears to be bound together by minute, blue-stained, high-cation exchange capacity clay particles whose small size indicates them to be smectite. It appears that for this clay the standard dispersion procedure is ineffective, and much of the clay is therefore unlikely to settle as expected in the hydrometer. An extended programme of testing suggests that soils which are not effectively dispersed by the standard procedure are not uncommon, and for estimating heave potential, hydrometer results may be deceptive and may lead to problems.

VARIABILITY OF SOIL PROPERTIES

A rather neglected feature of soils which is being examined by the CUT soils research group is variability. Current testing methods tend to hide variability of soil properties by specifying thorough mixing of samples before testing. This should give a good idea of the average properties of a soil sample, but it ensures that variability in those properties will be hidden.

Investigations by the SMRG suggest that variability of at least some soil properties may have a fractal distribution in at least some cases. A mathematical fractal distribution has patterns

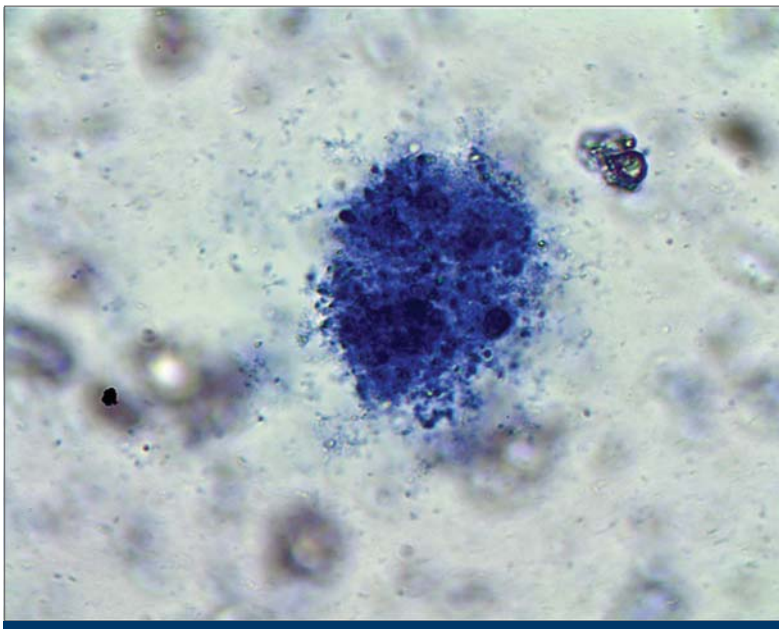

Photo 4: The large agglomeration of silt and clay-sized particles appears to be bound together by minute, blue-stained, high-cation exchange capacity clay particles

Photo 5: Research assistant, Alan, with the new conmatic auto consolidation apparatus recently acquired to conduct oedometer tests

which repeat, with only minor differences, at different scales from very small to very large. Real-life fractal distributions differ from true mathematical fractal distributions in that the patterns tend to show somewhat larger differences at different scales, and there are limited ranges of scale over which the fractal pattern repeats. The significance of identifying such a distribution pattern is that it is reasonably quick and easy to measure variations on a small scale, for example on the scale of a test pit. It is far more time-consuming to measure on a larger scale, such as that of a major construction site. Tests performed at the SMRG laboratory suggest that heave potential may have a fractal distribution on a range of scales large enough to be significant for design purposes.

The range of variability of heave potential is surprisingly large between different soils, and it is not easy to identify which soils have large variability and which have small variability without performing tests aimed at assessing this. Some highly plastic soils, which could be expected to be very troublesome from a heave perspective, show very small coefficient of variation (CoV) – of the order 2 or even less. Others show such large CoVs that there are clearly very real risks in using standard procedures to assess them. CoVs of 15 to 20 are not uncommon, and CoVs of more than 30 have been found for some soils. The problem here is that a soils test from one place in a test pit might give a PI of 12, suggesting a material which should give few problems for foundations, whereas an apparently identical sample from a short distance away in the same test pit might give a PI of 40. This aspect of soil property variability is now being taken into account in all the investigations being undertaken by the SMRG.

MOVEMENT OF MOISTURE UNDER A LIGHT-STRUCTURED HOUSE

The South African government's attempts to provide affordable, subsidised housing for the very poor has suffered from a large number of structural failures, many due to heaving foundations. These houses are particularly susceptible to damage by heaving clay, because they are exceptionally light and clay can lift them very easily. Rational design requires knowledge of the pattern of heave which will occur under the foundation. The pattern of heave depends on the pattern of moisture movement. Currently available methods of rational design rely on assumptions about the shape of the mound which will develop due to moisture movement under the foundation. The shape assumed is largely guided by measurements made on test foundations. Instrumentation has been installed under a government subsidy house in the Free State where the moisture movement is being monitored. These measurements suggest that currently accepted patterns of heave are unlikely to provide good guidance for foundation design. The instrumentation used in this investigation has proved itself convenient and reliable, and it is hoped that it will be possible to instrument several other light structures in order to work towards a general modelling procedure. This should enable reliable predictions of the moisture conditions which need to be designed for in the general case, which in turn should allow reliable and economic design of a wide range of raft foundations with the prospect of fewer failures.

EXPANSIVE POWER OF CLAY

The SMRG is looking into quick and convenient procedures to measure the pressures against which clays can expand and the speed at which this expansion takes place. This should give a better indication of the heave potential of clay than indirect measures such as the PI and clay fraction, and should overcome some of the difficulties which have led to the oedometer losing favour as a preferred testing tool for expansive clay.

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Are you smarter than a student?

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Karl Terzaghi made the following statement in a speech: "Students take to gadgets and neat little mathematical procedures like ducklings to water" (Hanson 1984). If one assumes that geotechnical research essentially produces such "gadgets and neat little mathematical procedures", then there is no shortage of interest for the modern student. This is evidenced by the exponential increase in Google Scholar search hits for terms related to geotechnical engineering (Figure 1). Terzaghi therefore hoped that university lecturers would "… educate a generation of foundation engineers who retain their common sense and their sense of proportion in spite of having been fed a dangerous drug – the drug of higher learning" (Hanson 1984).

The terms "common sense" and "sense of proportion" are essentially interchangeable with the term "engineering judgement", which Vick (2002) defines as "a sense of what is important". Jennings was perhaps more to the point when he said during a lecture, "… engineering judgement involves assembling all the facts you can, doing all the calculations you can, and then, on the basis of a good bottle of brandy and a good night's sleep, making your decision" (Caldwell 2015). Jennings' comment points to the core of judgement, which is the ability to match data, hypotheses, arguments and evidence, or simply being able to assign meaning to a calculated quantity (brandy aside!) (Vick 2002).

An exercise was recently conducted at the University of the Witwatersrand's School of Civil and Environmental Engineering to help students gain a "sense of proportion" for slope stability – a key component of geotechnical engineering practice. The aim of the exercise was to wean students from the temptation to fly straight for neat formulas that are easy

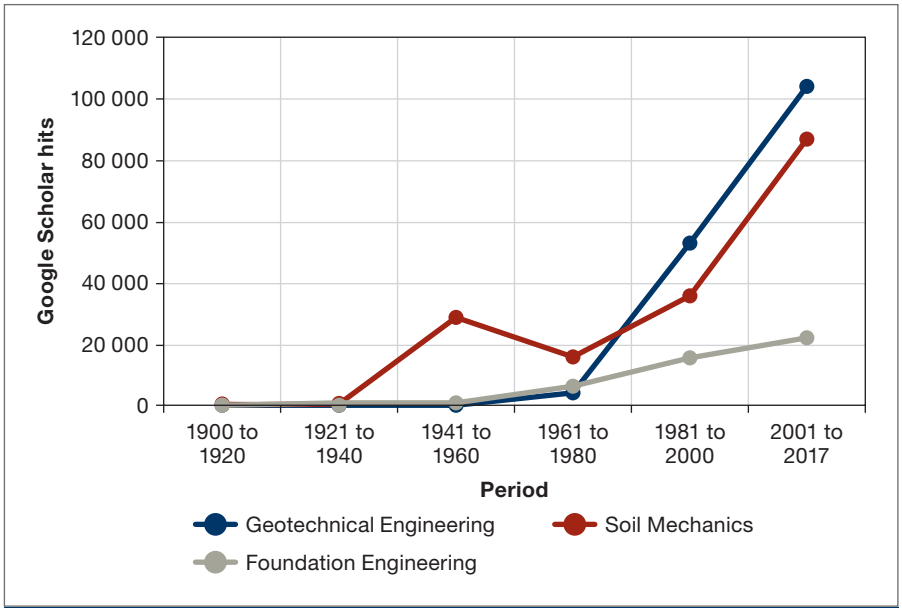

Figure 1: Google Scholar search hits for terms related to geotechnical engineering

Table 1: Strength scenarios

Undrained strength scenarios					
Descriptor	Very soft	Soft	Firm	Stiff	Very stiff
Undrained shear strengths, s_u (kPa)†	15	30	60	120	225
Drained strength scenarios					
Descriptor	Very loose	Loose	Medium dense	Dense	Very dense
Friction angle, φ' (°)†‡	25	30	35	40	45

† Unit weight, γ = 20 kN/m³ used in all cases
‡ Cohesion intercept (c) = 0 kPa in all cases

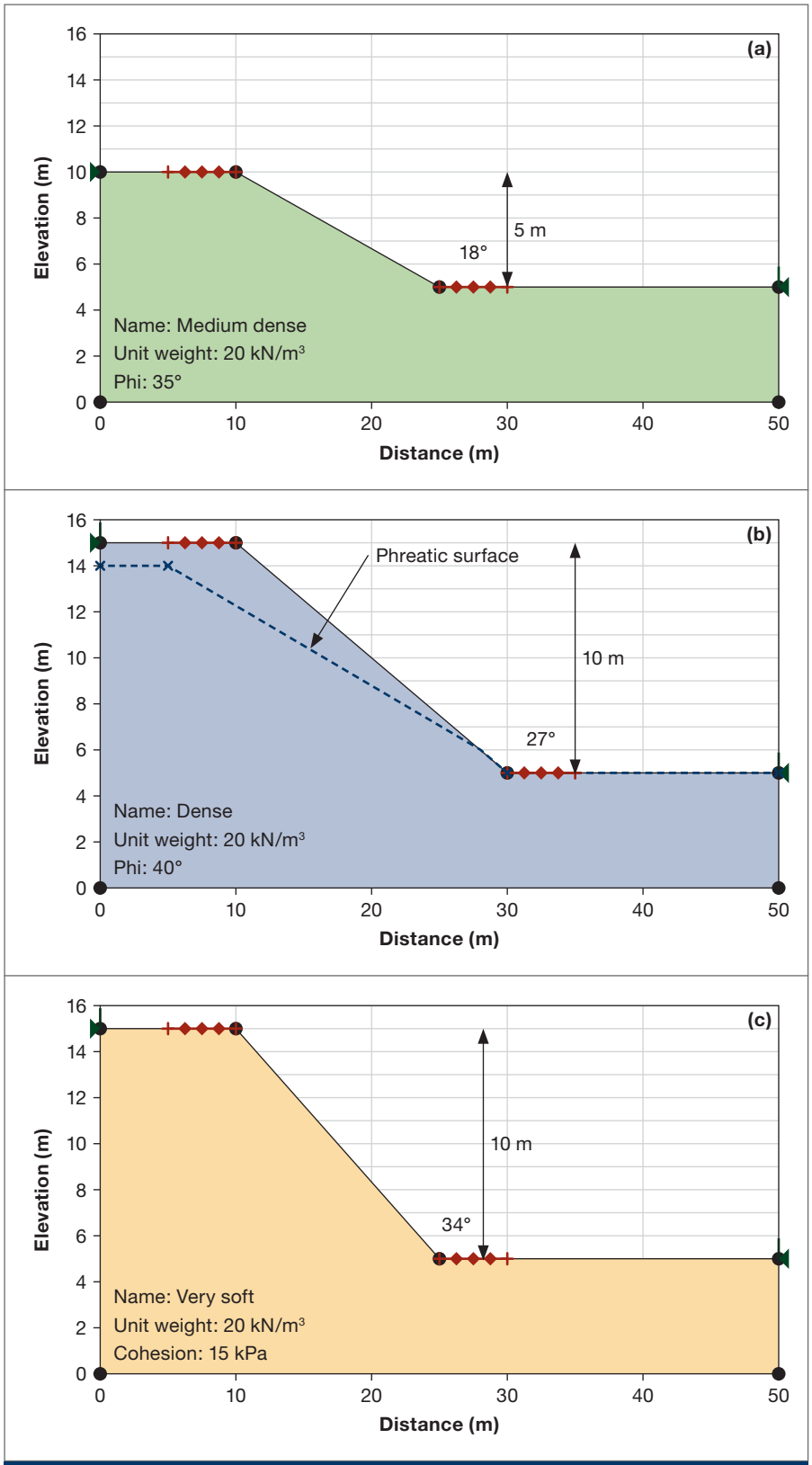

Figure 2: Example slopes: (a) drained without a phreatic surface, (b) drained with a phreatic surface, (c) undrained

This type of slip surface was chosen as typical of hand calculations set for students which involve slip circles that pass through the crest of the slope and exit beyond the toe.

to rote-learn. Through this approach students would develop pattern recognition to get an initial feel for the factors that influence slope stability. The exercise was also extended to practising engineers to see if their field experience showed any advantage over undergraduate students. This article details the assignment, and reports preliminary results from the exercise.

GETTING A "SENSE OF PROPORTION" FOR SLOPE STABILITY

Early in their first course on geotechnical engineering, prior to any teaching on slope stability, students were required to assess the stability of slopes based purely on a visual representation of the slope and material. Based on a representation of the essential aspects of slope stability – the geometry, soil strength and pore pressure regime – students would have to judge whether the slope was stable or not. This exercise was not a substitute for the teaching of slope stability, but rather an aide to the learning of slope stability.

GENERATING SLOPE STABILITY PROBLEMS

To generate a data set of slope stability problems, 150 slopes were assessed using the Slope/W Limit Equilibrium Program (GEO-SLOPE 2007). Geometries were based on five slope angles (1v:0h, 1v:1h, 1v:1.5h, 1v:2h and 1v:3h) and two slope heights (5 m and 10 m). Three different strength scenarios were assumed: undrained, drained without a phreatic surface and drained with a phreatic surface (Table 1). The phreatic surface was defined with a piezometric line. The piezometric line was defined as an initially horizontal line 1 m below the crest, which then dips down when it is 5 m from the edge of the crest, intersecting the slope 1 m up from the toe; it then followed the slope and ground surface. Three typical examples are given in Figure 2.

Factors of safety (fos) were computed using the Bishop (1955) method of analysis, and slip surfaces were determined with the entry and exit slip surface option. The entry area was defined as 5 m back from the crest edge and the exit area as 5 m from the toe. This type of slip surface was chosen as typical of hand calculations set for students which involve slip circles that pass through the crest of the slope and exit beyond the toe. A minimum slip surface depth of 2 m was set, also to reflect typical hand calculations. As the aim of the assignment was not to predict actual fos, slopes were labelled as either unstable (fos lower than 0.9), potentially unstable (fos between 0.9 and 1.3) and stable (fos greater than 1.3). These boundaries are considered to be in line with engineering practice (USACE 2003).

Two objections may be raised concerning how the slip surfaces were defined. The first is specifying a minimum slip surface depth of 2 m. This may result in shallow slips for drained scenarios without a phreatic surface being overlooked. The second objection is using the entry and exit slip surface option for cases involving drained parameters with a phreatic surface. This results in shallow localised failures, where the phreatic surface daylights, being overlooked. To judge the two objections, all drained slope scenarios were also analysed using the auto locate slip surface option with a minimum slip surface of 0.1 m.

Figure 3 compares fos determined with the less conservative slip surface definition used in the assignment to fos determined with the more conservative auto locate definition. Five slopes that were labelled stable in the assignment could potentially be unstable using the more conservative slip surface definition (blue box in Figure 3). Six slopes defined as potentially unstable could be unstable using the more conservative slip surface definition (green box in Figure 3). The five cases without a phreatic surface are perhaps less problematic, as such thin surface slides are unlikely to occur in reality due to soil suctions or vegetation (GEO-SLOPE 2012). For the cases with a phreatic surface, the two labelled as stable, when in fact they may potentially be unstable, are the most misleading. Nevertheless, it was hoped that the overall learning experience would have benefited the students, despite these potential questions of judgement.

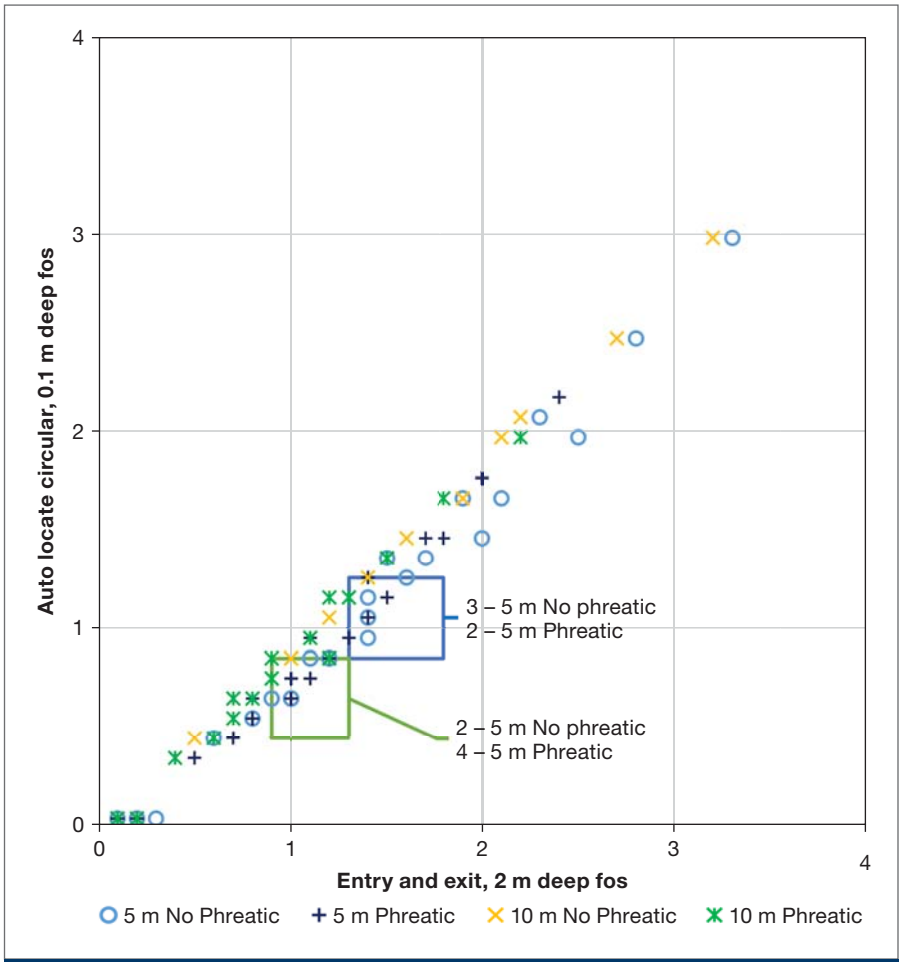

Figure 3: Comparison of factors of safety for different failure mechanisms

ASSIGNMENT SCHEMA

The assignment schema involved three phases: (i) an initial base line test, (ii) a training phase, and (iii) a final test. All phases were carried out on the Sakai online platform used by the University of the Witwatersrand. Students were presented with a single slope at a time (see Figure 2 for examples) and were required to select whether the slope was unstable, potentially unstable or stable. The number of slopes, sequence of slope types, answer feedback and time limit were varied between the three phases.

The initial base line test was used to establish a base from which to gauge progress. In this test, students were required to assess the stability of 30 randomly selected slopes from the data set with no prior knowledge of what the assignment entailed. A time limit of 20 minutes was set and the assignment was carried out under exam conditions in the School's computer laboratory. After the test only total score marks were given with no feedback of performance on individual questions.

The second training phase involved three separate assignments (termed training sets) in which students progressively worked through all 150 slopes. After selecting whether they felt a slope was unstable, potentially unstable or stable, the appropriate solution (Figure 4) was given and students had to type in the actual fos to progress to the next question. The sequence of slope types in each of the three training sets became increasingly random (Table 2) and harder. The time limit for completing a training set and the number of times it could be repeated were also specified (Table 2). Students could complete the training sets on any computer with internet access.

The last phase of the assignment schema was the final test. In this final

Table 2: Training set details

Training set	Sequence of slopes	Time and repetition limits
1	Ordered first by strength scenario (undrained, followed by drained without a phreatic surface and then drained with phreatic surface), then by slope angle (steepest to flattest) followed by strength (weakest to strongest).	1.5 hours Unlimited submissions
2	No order to strength scenario or slope angle. However, for a given strength scenario and slope angle, slopes were presented in order of strength (weakest to strongest).	1 hour No repetition allowed
3	No order to either strength scenario, slope angle or strength.	1 hour No repetition allowed

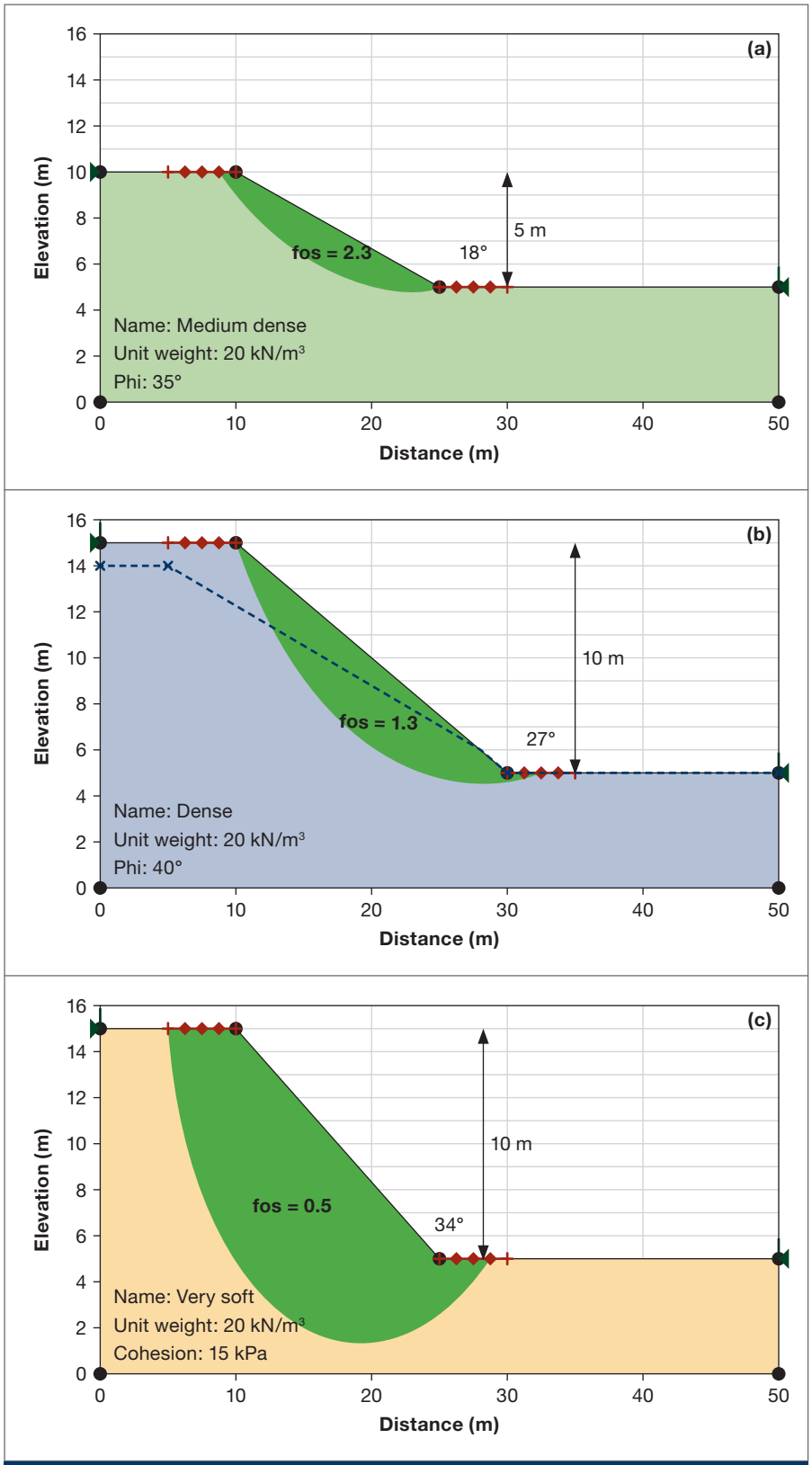

Figure 4: Example slope solutions: (a) drained without a phreatic surface, (b) drained with a phreatic surface, and (c) undrained

Whether this exercise will help wean students from neat mathematical procedures is yet to be seen, especially as many students queried or complained about the lack of an appropriate equation to be used.

test, students were required to assess the stability of 42 slopes. This included the initial 30 slopes and an additional 12 slopes to ensure an equal distribution between strength scenarios (i.e. undrained, drained without a phreatic surface and drained with a phreatic surface), and an equal distribution between unstable, potentially unstable and stable slopes. Slopes were randomly presented. A time limit of 20 minutes was set and the assignment carried out under exam conditions in the School's computer laboratory. After the test only total score marks were given with no feedback of performance on individual questions.

In addition to the students, the final test was also made available to various engineers practising within the geotechnical field. Two non-engineering staff of a geotechnical consultancy also participated in the test. Whilst the number of slopes, the sequence of slopes and time limit were kept the same, the test was not carried out under exam conditions within the School's computer laboratory, but rather from the participants' personal computers.

DID IT MAKE A DIFFERENCE?

Results from the engineering judgement exercise are depicted in Figure 5. Starting from the left-hand side of Figure 5, the probability of obtaining a particular score by guessing is shown with a solid black line. The calculated probabilities are based on the binomial theory, which assumes that both the actual answers and the chosen answers are completely random. It is unlikely that this is the real case, but the assumption is adequate for the purposes of this discussion. Next in Figure 5 are the scores obtained by all students before and after the training sets, depicted with open blue and green symbols respectively. Symbols stacked horizontally show the number of students who obtained the same score. On the right-hand side of Figure 5, scores obtained by industry participants are plotted relative to their years of experience. Industry participants were also grouped into different categories (see the key in Figure 5) for the purposes of averaging. The long dash line shows the average score and the short dash lines show the interquartile range obtained by each group.

From Figure 5 it is apparent that students performed better in the base line test (i.e. before training) than if they had simply guessed. It is also clear that students performed much better in the final test after the training sets. Considering individual students, the percentage increase in scores varied between 206% and –17% (i.e. a decrease) and was on average 33%. It has been noted

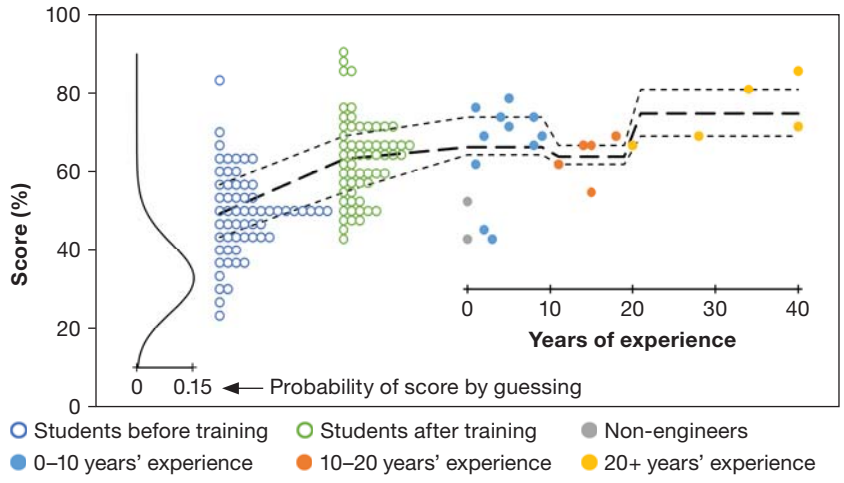

Figure 5: Results from engineering judgement exercise

that key to developing engineering judgement is repetition, as this enables one to recognise patterns, although for some individuals repetition does not always result in improved judgement (Vick 2002). Through repetition, the training sets enabled students to recognise patterns, which improved their performance.

Considering the performance of industry participants (Figure 5), as expected the two non-engineers scored similarly to the students prior to training. The scatter and average of results for engineers with 0 to 10 years' experience were similar to that of students after training. However, most of the engineers with 0 to 10 years performed better than the student average. Scores for engineers with 10 to 20 years' experience are perhaps anomalous, as their performance was lower than that of engineers with 0 to 10 years' experience. This may be due to a greater measure of conservatism (or overconfidence) amongst engineers with 10 to 20 years' experience. Scores for engineers with more than 20 years' experience show that perhaps with age comes wisdom.

So, did the exercise make a difference in students' ability to judge slope stability? Did the engineering judgement exercise help students to gain a "sense of proportion"? The increase in student performance between the base line test and the final test suggests that students did gain a feel for the stability of a slope. This feel was based purely on a judgement of the essential aspects of slope stability – the geometry, soil strength and pore pressure regime – and no calculations. This data also shows the importance of repetition in learning and gaining understanding of fairly complex engineering problems, and being able to make experimental judgements of solutions. Whether this exercise will help wean students from neat mathematical procedures is yet to be seen, especially as many students queried or complained about the lack of an appropriate equation to be used.

Lastly, we may ask if practising engineers are smarter than students? Well, perhaps smarter than untrained students, but the performance of trained students was on average very similar to that of practising engineers. I will let you be the judge …

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SAICE Training Calendar 2017

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
GCC 2015 (Third Edition)	20–21 April 2017	Cape Town	SAICEcon16/01869/19	Benti Czanik	cheryl-lee@saice.org.za
	15–16 May 2017	Durban			
	18–19 May 2017	Pietermaritzburg			
	19–20 June 2017	Port Elizabeth			
	22–23 June 2017	East London			
	17–18 July 2017	Pretoria			
	20–21 July 2017	Polokwane			
	7–8 September 2017	Midrand			
	18–19 September 2017	Bloemfontein			
	9–10 October 2017	Kimberley			
GCC 2015 and GCC 2010 Differences	16 August 2017	Durban	SAICEcon16/01890/19	Benti Czanik	dawn@saice.org.za
	18 October 2017	Cape Town			
Project Management of Construction Projects	20–21 July 2017	Midrand	SAICEcon15/01754/18	Neville Gurry	cheryl-lee@saice.org.za
	9–10 October 2017	Cape Town			
Technical Report Writing	29–30 May 2017	East London	SAICEbus15/01751/18	Les Wiggill	cheryl-lee@saice.org.za
	31 May–1 June 2017	Port Elizabeth			
	22–23 May 2017	Polokwane			
	26–27 June 2017	Nelspruit			
	27–28 July 2017	Durban			
	3–4 August 2017	Bloemfontein			
	28–29 September 2017	Midrand			
Structural Steel Design to SANS 10162-1-2005	14 August 2017	Durban	SAICEstr15/01726/18	Greg Parrott	cheryl-lee@saice.org.za
	28 September 2017	Midrand			
	23 October 2017	Cape Town			
Reinforced Concrete Design to SANS 10100-1-2000	15 August 2017	Durban	SAICEstr15/01727/18	Greg Parrott	cheryl-lee@saice.org.za
	29 September 2017	Midrand			
	24 October 2017	Cape Town			
Practical Geometric Design	5–9 June 2017	Cape Town	SAICEtr16/01954/19	Tom Mckune	dawn@saice.org.za
	6–10 November 2017	Midrand			
Business Finances for Built Environment Professionals	8–9 June 2017	Midrand	SAICEfin15/01617/18	Wolf Weidemann	dawn@saice.org.za
	9–10 November 2017	Midrand			
Handling Projects in a Consulting Engineer's Practice	5–6 June 2017	Midrand	SAICEproj15/01618/18	Wolf Weidemann	dawn@saice.org.za
	6–7 November 2017	Midrand			
Leadership and Management Principles and Practice in Engineering	16–17 August 2017	Midrand	SAICEbus15/01784/18	David Ramsay	dawn@saice.org.za
Leadership and Project Management in Engineering	6–7 September 2017	Durban	SAICEbus16/01950/19	David Ramsay	dawn@saice.org.za
	4–5 October 2017	Cape Town			
Water Law of South Africa	9–10 May 2017	Durban	SAICEwat16/01955/19	Hubert Thompson	dawn@saice.org.za
	25–26 July 2017	Cape Town			
	19–20 September 2017	Midrand			
Earthmoving Equipment, Technology and Management for Civil Engineering and Infrastructure Projects	17–19 May 2017	Port Elizabeth	SAICEcon15/01840/18	Prof Zvi Borowitsh	dawn@saice.org.za
	25–27 October 2017	Midrand			

SAICE Training Calendar 2017

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
The Legal Process Dealing with Construction Disputes	30–31 May 2017	Port Elizabeth	SAICEcon16/01956/19 SACPCMP/CPD/15/010	Hubert Thompson	dawn@saice.org.za
	1–2 August 2017	Midrand			
	15–16 August 2017	Cape Town			
	5–6 September 2017	Durban			
	7–8 November 2017	Bloemfontein			
Sanitary Drainage Systems for Buildings	18 May 2017	Midrand	SAICEwat15/01957/18	Vollie Brink	dawn@saice.org.za
	10 October 2017	Midrand			
Claims Cast in Concrete	15–16 May 2017	Midrand	SAICEcon15/01759/18	Bruce Raath	cheryl-lee@saice.org.za
Durability and Repairs of Concrete Structures	14–15 August 2017	Midrand	SAICEcon15/01757/18	Bruce Raath	cheryl-lee@saice.org.za
Legal Liability Occupational Health and Safety Act (OHSA)	Date to be confirmed	TBC	SAICEcon17/02038/20	Cecil Townsend Naude	dawn@saice.org.za
	Date to be confirmed	TBC			
	Date to be confirmed	TBC			
Construction Regulations from a Legal Perspective	Date to be confirmed	TBC	SAICEcon17/02037/20	Cecil Townsend Naude	cheryl-lee@saice.org.za
	Date to be confirmed	TBC			
	Date to be confirmed	TBC			
Principles and Practices of Facility Management for Engineers	18–19 July 2017	Midrand	SAICEbus17/02042/20	Wynand Dreyer / Lwandiso Mgwetyana / Shane Verster	dawn@saice.org.za
	19–20 September 2017	Durban			
	14–15 November 2017	Cape Town			

SAICE / South African Road Federation (SARF)

Asphalt: An Overview of Best Practice	30–31 May 2017	Gauteng	SAICEtr15/01806/18 SARF15/5001/18	J Onraet	sybul@sarf.org.za / tshidi@sarf.org.za
	19–20 September 2017	Polokwane			
Assessment and Analysis of Test Data	4–5 July 2017	Bloemfontein	SAICEtr15/01805/18 SARF14/0001/17	R Berkers	sybul@sarf.org.za / tshidi@sarf.org.za
	5–6 October 2017	Cape Town			
Stormwater Drainage	Date to be confirmed	Durban	SAICEtr15/01808/18 SARF12/0107/15	C Brooker Matt Braune Alaster Goyns	sybul@sarf.org.za / tshidi@sarf.org.za
Concrete Road Design and Construction	26 July 2017	Cape Town	SAICEtr15/01802/18 CSSA-N-2013-08	B Perrie Dr P Strauss	sybul@sarf.org.za / tshidi@sarf.org.za
	30 August 2017	Durban			
	12 September 2017	Midrand			
Traffic Signals Design and Optimisation – with special emphasis on BRT	19–20 June 2017	Gauteng	SAICEtr15/01803/18 SARF14BRT09/17	Dr John Sampson	sybul@sarf.org.za / tshidi@sarf.org.za
	29–30 August 2017	Bloemfontein			
Construction of G1 Bases	19 September 2017	Port Elizabeth	SAICEtr15/01809/18 SARF14/9103/17	E Kleyn	sybul@sarf.org.za / tshidi@sarf.org.za

SAICE / Mentoring 4 Success

One-day Workshop – Foundations in Structured Mentoring in the Workplace	13 June 2017	Gauteng	SAICEbus16/01894/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
	12 September 2017	Gauteng			
Mentors Masterclass in Engineering and Construction	13–14 June 2017	Gauteng	SAICEcon14/01675/17	Philip Marsh / Celestine Jeftha	info@m4s.co.za
	12–13 September 2017	Gauteng			

Candidate Academy

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
Road to Registration for Candidate Engineers, Technologists and Technicians	6 June 2017	Midrand	CESA-861-05/2019	Allyson Lawless	lizelle@ally.co.za
	22 June 2017	Upington			
	24 July 2017	Durban			
	12 September 2017	Midrand			
Pressure Pipeline and Pump Station Design and Specification – a Practical Overview	25–26 May 2017	Midrand	CESA-872-05/2019	Dup van Renen	lizelle@ally.co.za
	20–21 September 2017	Cape Town			
	11–12 October 2017	Midrand			
Getting Acquainted with Geosynthetics in Soil Reinforcement	16–18 May 2017	Midrand	SAICEgeo14/1627/17	Edoardo Zannoni	lizelle@ally.co.za
Road to Registration for Mature Candidates	31 May 2017	Durban	CESA-948-11/2019	Peter Coetzee Stewart Gibson	lizelle@ally.co.za
	27 July 2017	Midrand			
	20 September 2017	Cape Town			
	2 November 2017	Durban			
	23 November 2017	Midrand			
Getting Acquainted with Road Construction and Maintenance	24–25 July 2017	Midrand	CESA-870-05/2019	Theuns Eloff	lizelle@ally.co.za
Road to Registration for Mentors, Supervisors and HR Practitioners	23 May 2017	Midrand	CESA-862-05/2019	Allyson Lawless	lizelle@ally.co.za
Getting Acquainted with General Conditions of Contract for Construction Works (GCC 2015)	8–9 June 2017	Midrand	CESA-873-05/2019	Theuns Eloff	lizelle@ally.co.za
	3–4 August 2017	Cape Town			
	14–15 August 2017	Durban			
	23–24 October 2017	Midrand			
Getting Acquainted with Sewer Design	13–14 June 2017	Midrand	CESA-871-05/2019	Peter Coetzee	lizelle@ally.co.za
	6–7 September 2017	Cape Town			
	21–22 November 2017	Durban			
Getting Acquainted with Basic Contract Administration and Quality Control	17–18 August 2017	Midrand	CESA-864-05/2019	Theuns Eloff	lizelle@ally.co.za

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